CONCRETE-STEEL BUILDINGS

W. N. TWELVETREES

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CONCRETE-STEEL BUILDINGS

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BEING A COMPANION VOLUME TO THE TREATISE ON "CONCRETE-STEEL"

BY

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PREFACE

IN *Concrete-Steel* the author has already dealt with the distinctive characteristics of reinforced concrete, discussing the principles underlying construction in that material, and stating simple rules for its application to the design of the primary members employed in structures of nearly every kind.

Since the publication of that work a remarkable change has taken place in the attitude of engineers, architects, and others with regard to the new structural material, which is now being adopted on an extensive scale in the United Kingdom by private individuals, industrial firms, railway and dock companies, municipal authorities, and Government Departments. In view of the widespread desire for information on the subject, and in response to the desire expressed by readers of the preceding book, the author now presents detailed particulars of some buildings designed for use, in Great Britain, on the Continent, and in America, as dock sheds, railway goods stations, locomotive sheds, warehouses, manufactories, workshops, flour mills and granaries, hospitals, hotels, residences, churches, theatres, and public halls.

Some of the works described and illustrated are specially noteworthy for their size, others in respect of their great strength, others again for the manner in which difficult problems have been solved, and all of them as indicat-

ing the adaptability of concrete-steel to structural requirements of the most varied description.

In order to increase the value of the book to practical men, an Index has been prepared containing copious references to the details of the buildings described, as well as to data concerning the proportions and consistency of the concrete used, the amount of the reinforcement, the strength of materials employed, the results of tests, contractors' plant and methods of practical construction, and other technical matters.

With the object of emphasising the necessity for correct design, competent supervision, and skilful construction, the concluding chapter is devoted to "Some Mishaps and their Lessons," and an Appendix has been added as a guide to the places where typical examples of concrete-steel structures are to be found in all parts of the United Kingdom.

While recognising the fact that it would be impossible in a single volume to deal completely with all the concrete-steel buildings erected even in this country, the author hopes that the records here presented may be of service to those who are already convinced of the merits possessed by concrete-steel, and further, that they may have the effect of arousing the interest of those who have not yet realised its peculiar suitability as a material of construction, or have not been convinced of the ease with which it can be applied to building and engineering construction.

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CONCRETE-STEEL BUILDINGS

CHAPTER I

Transit Sheds at Manchester Docks

1. The Dock Extension Scheme.—So rapidly has traffic increased on the Manchester Ship Canal that continual additions have been necessary to the accommodation originally provided at the inland terminal. For some time past the old docks have been inconveniently crowded and the warehouse capacity has been much overtaxed. Hence, four or five years ago, it became absolutely essential to provide additional quay space for vessels, and facilities for dealing with merchandise brought into and sent out from the port.

The works then decided upon included the construction of new docks and quays, the erection of an extensive range of buildings for use as transit sheds, and the laying out of railways, sidings, and roads. The site secured was formerly occupied by the Manchester Race Course Company, and covers an area of some 150 acres. About $21\frac{1}{2}$ acres of this will be devoted to the new docks, and the remaining portion to the quays and the other auxiliary works.

The first dock, officially known as Dock No. 9, with an area of $15\frac{1}{2}$ acres, has already been completed; while the

FIG. 1.—Manchester New Dock and Transit Sheds (North Front).

FIG. 2.—Manchester Dock Transit Sheds (South Front).

other, Dock No. 10, which will have an area of 6 acres, remains to be built.

2. The New Buildings.—We have to deal here only with the series of five transit sheds built in connection with Dock No. 9. These sheds, having a frontage of about 2,250 ft., extend almost from end to end of the quay along the south side of the new dock, and have a uniform width of 110 ft. Fig. 1 is a view showing part of the new dock and some of the warehouses. Fig. 2 is a view of the other side of the buildings, but does not include more than about one-fourth of the entire range.

In view of the disastrous fires which have occurred in various ports where combustible buildings existed, the directors of the Manchester Ship Canal wisely determined that their new buildings should embody the most approved system of fire-resisting construction. Concrete-steel is a material admirably complying with the required conditions, and one that has already been applied on an extensive scale to the building of warehouses on the Thames, at Southampton, and at other British ports.

The new sheds at the Manchester docks are built entirely of concrete-steel on the Hennebique system, the general designs having been prepared by Mr. W. H. Hunter, M.Inst.C.E., Chief Engineer to the Manchester Ship Canal, and the details of construction by Mr. L. G. Mouchel, M.Soc.C.E. (France), of Westminster. The building contractors were Messrs. Lovatt & Brueder, of Wolverhampton.

The middle shed is 450 ft. long, and the other four 425 ft. long each, the whole series being connected by gangways or bridges joining the upper floors and the roofs of adjacent buildings. Fig. 3 is a part elevation of one of the 425-ft. sheds. This drawing shows the south front facing the railway sidings, the north front on the opposite side facing the new dock.

Although known as sheds, it will be seen that the buildings are of more important character than is suggested by the designation applied to them. Each shed comprises three floors and a flat roof, virtually constituting a fourth floor.

The height from the ground floor to the under side of the main beams of the first floor is 14 ft. 3 in., the height of the first storey is 8 ft. 4 in., and that of the second storey 8 ft. 7 in. These measurements are taken in each case to the under side of the floor beams. The height of the sheds from ground level to the roof is 45 ft., and to the ridges of the towers 51 ft. 3 in. All the roofs are flat, so as to be available for the storage of packing-cases, crates, and merchandise not liable to injury by the weather, the collective floor area provided by the series of five sheds being about 950,000 sq. ft., or nearly 22 acres.

The floors of the first storey are prolonged to form a balcony 12 ft. wide along the south front of each building, except where interrupted by five towers, of which two are shown in Figs. 3 and 4. Another balcony, with a width of 4 ft., runs along the north front. These balconies are indicated in broken lines on Fig. 4, which is a half-plan of one 425-ft. shed. The two towers in Fig. 3 are used for hoisting

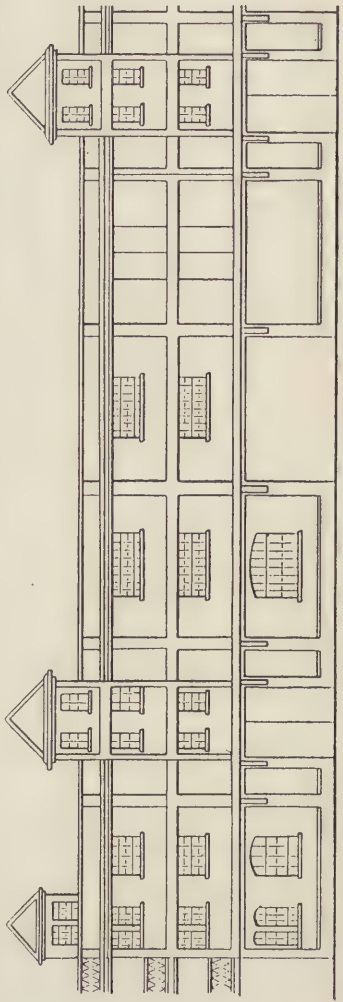


FIG. 3.—Part Elevation of 425-ft. long Transit Shed.

purposes, and the turret is a small building on the roof, a similar turret being situated above each staircase.

Construction was commenced in July 1903, and finished early in 1905 about six months in advance of the contract time, the total cost of the sheds, exclusive of fittings and equipment, amounting to more than £150,000.

3. Concrete and Contractors' Plant.—The quantity of concrete used on the sheds alone exceeded 26,000 cubic yards, and over 5,000 tons of steel were employed as reinforcement. The number of workmen employed varied from 350 to 400, and of this number about 20, including foremen carpenters and cementers, were of French nationality.

Sand and gravel for concrete making were provided by material excavated during the construction of the new dock, and special plant was laid down for washing, sorting, and crushing the materials. Except for special details, the concrete was mixed in the proportions of 1 part of Portland cement, 2 parts of clean sharp sand, and 4 parts of washed gravel from $\frac{1}{8}$ in. to $\frac{3}{4}$ in. gauge. The moulds for the concrete necessitated the employment of more than 141,000 cubic feet of timber.

For the purpose of facilitating the erection of the superstructure a track was laid along the whole length of the sheds, front and back, upon which a travelling stage was placed, and moved from point to point as required. This stage was equipped with two concrete mixers of the Oehler type, each capable of making 40 cubic yards of concrete in ten hours.

A small portable crane, running upon a transverse set of rails, was used to charge the concrete mixers. A large crane, travelling upon the main track, was employed for hoisting concrete and steel to the various floors and roof of the sheds. This crane had a jib with a radius of 23 ft., and was capable of raising a load of $1\frac{1}{2}$ ton to a height of 50 ft. Fig. 5 is a photographic view showing the crane in the act of depositing concrete in the moulds during construction.

The general method of construction being the same for each of the five sheds, it is only necessary to describe in detail a typical section of one shed.

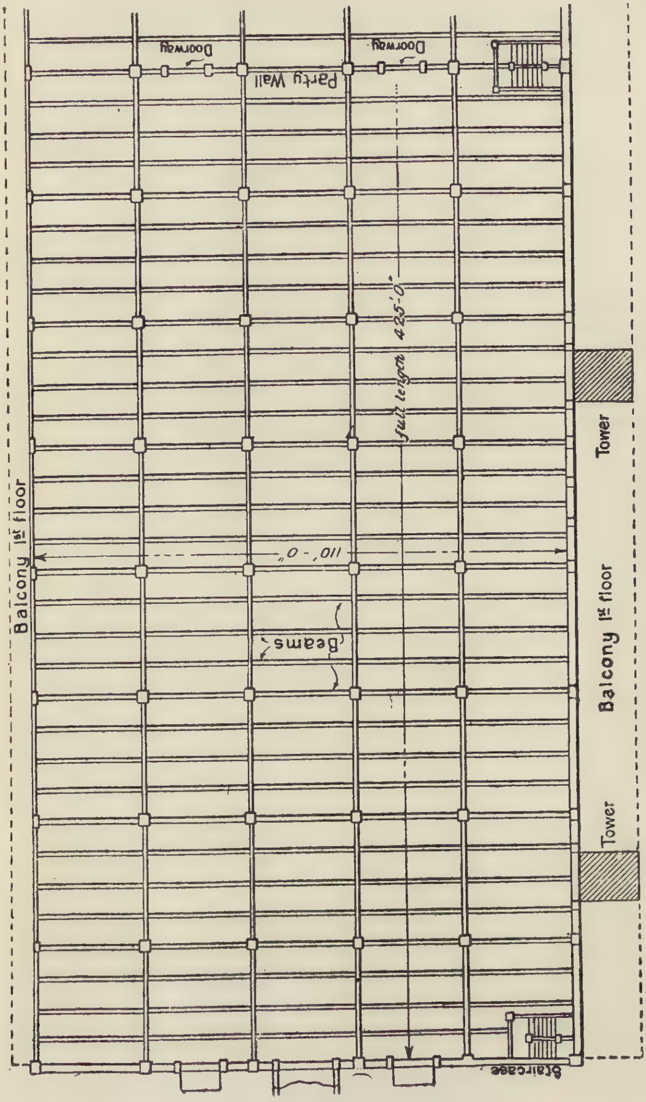


FIG. 4.—Half Plan of a 425-ft. long Transit Shed.

4. Foundations.—The foundations are provided by ninety-three concrete piers 6 ft. wide and 16 ft. deep, spaced 25 ft. apart centre to centre. These foundations are really extensions of the piers supporting the arched construction of the quay wall (see Fig. 1), each of them having a total length of 149 ft.,—37 ft. being beneath the quay and 112 ft. between the front and back boundaries of the new sheds.

Below the south front of the sheds the ends of the piers are connected by concrete arches 3 ft. thick, which were built without much excavation, as the earth was merely cut to the curve selected for the intrados of the arch, and, after being carefully dressed, was covered by concrete deposited upon it between timber shutters. By the adoption of this plan the expense of excavation and refilling was almost entirely avoided, as well as that of erecting and removing moulds. Moreover, the arches receive valuable assistance from the solid earth, which has far greater bearing power than material repacked into an excavation.

Owing to the low level of the concrete piers it was necessary to build brick footings (see Fig. 6) for the support of the column bases. These footings, about 7 ft. 9 in. square by 4 ft. high, are spaced 22 ft. apart, so that there are six of them along each of the concrete piers, which, as mentioned above, are spaced 25 ft. apart.

5. Column Bases.—Upon the brick piers are fixed cast-iron base plates 4 ft. in diameter by 9 in. high, the top of each being level with the surface of the ground floor. The bases afford bearing for the steel bars forming the vertical reinforcement of the columns, and over them are placed dome-shaped shields of cast iron. These, being filled with concrete, serve to hold the columns rigidly at the base, and at the same time to protect them from accidental injury.

6. Column Reinforcement.—The reinforcement of the columns consists of vertical bars connected, at intervals of about 6 in. apart, by $\frac{9}{16}$-in. diameter steel ties. The vertical bars are of $1\frac{7}{8}$ in. diameter, and 16 ft. 6 in. long so as to extend from the base plate to about 2 ft. above the

first floor, where they are connected with the vertical reinforcement of the section above.

Steel angle bars are provided outside the corners of every

FIG. 5.—Timber Moulds and Crane delivering Concrete.

column for the purpose of shielding the concrete from injury in the course of daily work in the sheds. These angles measure 4 in. by 4 in. by $\frac{3}{8}$ in., and extend from the floor almost to the ceiling of each storey. They are attached to the columns by 6-in. strips of hoop iron riveted to them at intervals, the ends of the strips being split for

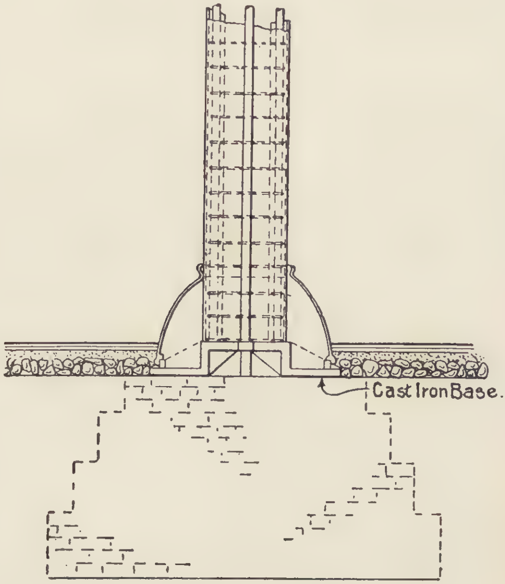


FIG. 6.—Column and Brick Footing.

a length of about 2 in., and opened in opposite directions so as to afford a secure bond.

7. Method of Moulding Columns.—When all the reinforcement and the angle plates had been erected and temporarily secured in position, the cast-iron bases and dome-shaped shields were filled in with concrete, which was well rammed. The column moulds were then erected and shored up, one side of each mould being left open.

The concrete was deposited in layers 6 in. thick, boards being fixed one at a time across the open side of the mould as each layer of concrete was finished. The concrete was mixed fairly wet, so as to make it easy to fill all spaces in the moulds, and it was rammed down until water rose to the surface, the ramming being performed by iron bars with a right-angled bend of 3 in. long at the end.

All the columns were built in a similar manner, but the details of construction necessarily varied in accordance with the loads to be carried.

8. Column Sections and Loads.—Fig. 7 contains cross sections of typical columns on the various floors of the sheds. Commencing at 20 in. square with ten vertical bars on the ground floor, the columns were reduced to 14 in.

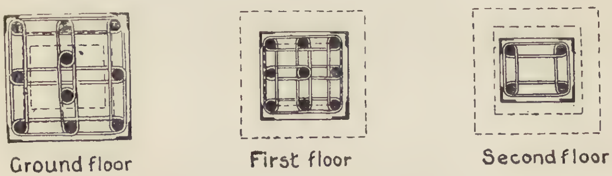


FIG. 7.—Cross Sections of Columns.

square with nine vertical bars on the first floor, and to 10 in. by 12 in. with four vertical bars on the second floor, while other columns supporting structures on the roof measure $6\frac{1}{2}$ in. by 12 in.

The columns on the ground floor were calculated for a normal load of 340 tons each, those on the first floor for a load of 226 tons each, and those on the second floor for a load of 113 tons each. These loads represent pressures of about 1,950 lb., 2,520 lb., and 2,100 lb. per sq. in. respectively for the three portions of each column, or an average of about 2,200 lb. per sq. in. of cross sectional area.

9. Column and Beam Connections. — To afford additional support for the main beams, the tops of the columns were extended to form a bracket on either side with a projection of 18 in. and a depth of 10 in. close to the column. The reinforcement of these brackets consists

of two $\frac{3}{4}$-in. diameter bars passing horizontally through the column, rising up at each end parallel to the under surface of the bracket, and continuing into the concrete of the beams above. Fig. 8 shows a column connection, part of a main beam, two cross sections of a secondary beam, and a section of the floor slab.

10. Floor Beams.—After the columns had been built up to the first floor level, and left to harden for about a week, the main and secondary beams were formed in timber moulds extending from column to column. The main beams, 12 in. wide by 18 in. deep, extend from end to end of each shed, tying the tops of the columns together and really constituting continuous girders.

The secondary beams, or joists, reach from column to column transversely across the building, forming panels 25 ft. by 22 ft., each of these rectangles being subdivided by three intermediate joists, supported at the ends by the main floor beams (see Fig. 4).

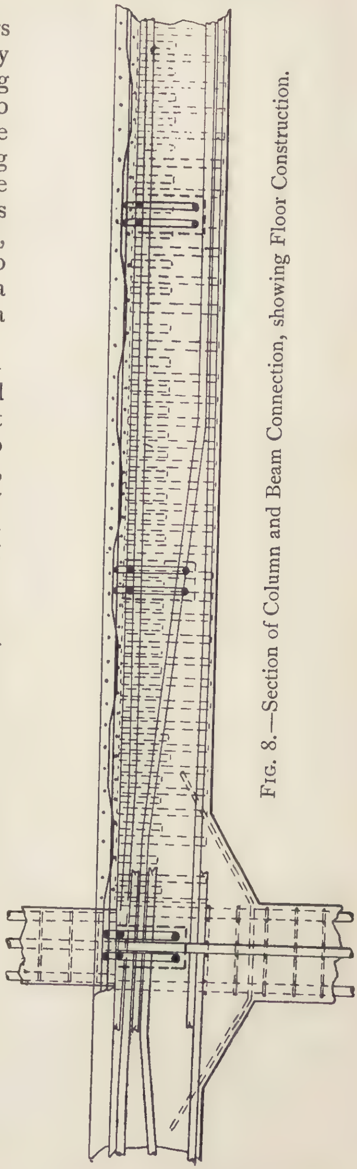


Fig. 8.—Section of Column and Beam Connection, showing Floor Construction.

11. Construction of Main and Secondary Beam.—The reinforcement in each of the main beams comprises nine longitudinal bars in rows of three abreast and numerous stirrups, also three abreast, placed 6 in. apart along the beam. Half of the stirrups pass under the lowest bars and half over the uppermost bars. Fig. 9 contains two sections of a typical main beam, one taken at the centre and the other near one of the supports where the middle three longitudinal bars are bent in an upward direction for the purpose of resisting tension between the supports and the points of contrary flexure.

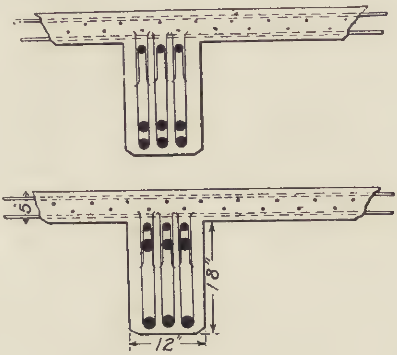


FIG. 9.—Section of Main Beams.

When the beam moulds had been erected and securely stayed the spaces between them were filled by timber panels laid level with the upper edge of the beam moulds, and the whole surface was coated with a layer of limewash before the commencement of concreting.

The first step in this operation was to spread a thin layer of concrete 1 in. thick along the bottom of the beam mould, and, after well ramming, to place the lower set of stirrups in position. These stirrups consist of No. 12 S.W.G. steel strip $2\frac{1}{2}$ in. wide by 43 in. long, and when bent into U-form the effective length was about 20 in., permitting the two ends of each stirrup to project about 3 in. through

the top of the beam, the projecting ends being afterwards incorporated in the concrete of the floor slab.

The stirrups, three abreast in the width of the beam, were spaced 6 in. apart longitudinally, and inside them were laid three bars of the longitudinal reinforcement, the diameter of these bars being $1\frac{3}{4}$ in. Next came a layer of concrete sufficient to cover the bars, and over this were laid three $1\frac{3}{4}$-in. bars, bent up towards the ends, so as to provide suitably for resisting tensile stress developed in the upper part of the beam section, between the supports and the points of contrary flexure. More concrete was then deposited, entirely covering the bent bars, and three $1\frac{1}{4}$-in. diameter bars were laid over it. The upper stirrups were next adjusted over the top bars and pushed down into the wet concrete, these stirrups being made of No. 13 S.W.G. steel strip $1\frac{3}{4}$ in. wide. The total length of metal in each upper stirrup was 19 in., giving an effective depth of about 8 in. The mould was finally filled up with concrete, and a period of four or five days was allowed for hardening before the addition of the floor slab.

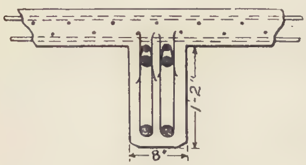


FIG. 10.—Section of a Secondary Beam.

A very similar course of procedure was followed in the construction of the secondary beams, which contain six bars of longitudinal reinforcement, four bars of $1\frac{1}{4}$ in. diameter near the bottom of the concrete, and two bars of $\frac{1}{2}$ in. diameter near the top. Fig. 10 is the cross section of a secondary beam near the support.

The ends of all the bars and stirrups were split and bent out so as to afford secure anchorage, and the round bars were jointed together at places where they overlapped. The extent of the overlap varied from about 12 in. to as much as 30 in., according to the strain coming upon the construction.

12. Floor Slab Construction.—After the concrete of the main and secondary beams had sufficiently set a 1-in. layer of concrete was spread over the centring, and upon

this the first series of rods of the reinforcement for the floor slab was laid out. These rods, ranging from $\frac{1}{4}$ in. to $\frac{5}{8}$ in. diameter, were spaced 12 in. apart in rows parallel to the secondary beams. A second 1-in. layer of concrete was spread over the rods, and the second set was then laid out, 6 in. apart, transversely to the first set.

Concrete was next deposited to the thickness of 2 in., in three strips, one 24 in. wide midway between and parallel with two adjacent joists, and one 12 in. wide next to each of the side joists of each floor panel. Thus in each span of 8 ft. between the joists there were three ridges and two hollows, over which the third series of rods was laid, 6 in. apart, each rod being bent to fit the contour of the concrete.

The hollows were next filled up, and enough concrete was added to cover the raised parts of the rods, the material being thoroughly tamped down. After this operation the fourth and last series of rods was laid at right angles to those previously fixed. A final layer of concrete was added and, after being rammed and levelled, made the slab 5 in. thick. As the work progressed the projecting ends of the beam stirrups became incorporated in the floor slab, which, extending continuously over the beams and joists, thereby increased the total depth of those members to 23 in. and 19 in. respectively. In every case the rods laid in the floor slab project over their supports by at least 6 in., and the ends are bent over to secure good anchorage. The details of the construction here described are represented in Fig. 8, and a view taken during the construction of the first floor will be found in Fig. 11, and one of the same floor viewed from below in Fig. 12.

13. Wall Construction.—All the walls of the sheds are built entirely of concrete-steel, and, as they carry no load whatever, it was not necessary to make them more than 4 in. thick. It should be noted, however, that the north fronts of the sheds have no walls, strictly speaking, but are provided with cast-iron sliding doors, fitted with rollers, running on top and bottom guides.

The method adopted in building the walls was to erect a network of thin steel rods between adjacent columns at

FIG. 11.—First Floor of a Shed under Construction.

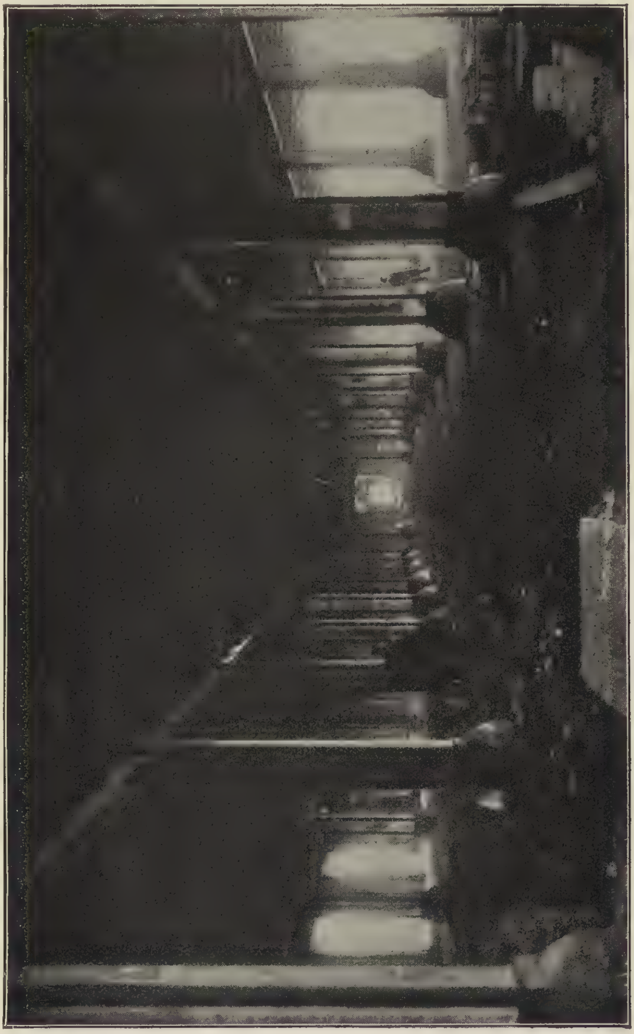


FIG. 12.—Interior of a Shed, showing Columns and Floor Beams.

the ends and back of each shed, and then to fill in concrete by the aid of suitable moulds, built up of boards as the work proceeded.

On the first and second floors an opening 12 ft. wide was left in the middle of the wall at each end of every shed, giving access to the gangways connecting the adjacent warehouses (see Figs. 3 and 4). Two other openings were provided at each end wall on either side of the gangway, as shown in Fig. 4, these being intended to facilitate the loading and unloading of vans and lorries, by means of 30-cwt. electrically driven hoists fixed on the roof.

14. Cantilever Towers and Balconies.—Reference

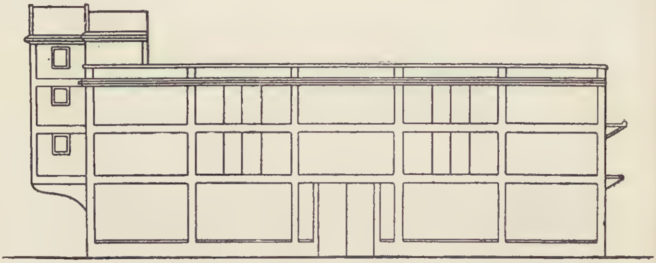


FIG. 13.—Manchester Dock Transit Sheds (East Elevation).

was made in Art. 2 to the five towers at the back of each shed. As shown in Fig. 13, they project a considerable distance beyond the face of the building, and extend from the level of the first floor to a height of some feet above the roof level, being supported on massive cantilevers of concrete-steel. The upper part of each tower is extended for about 10 ft. over the flat roof of the shed, and under the roof of each tower an electrically driven hoist is fixed on two 16-in rolled steel joists 22 ft. long. Concrete-steel columns of special design are built into the walls for the purpose of helping to support these joists.

The towers are perfectly open from the bottom to the roof, being intended for the hoisting of perishable goods under cover from vehicles below, or from one floor to

another, or for loading up vehicles with goods passing through or stored in the sheds. The towers are about 12 ft. square, and have 4-in. walls of concrete-steel, in which windows are provided on each storey, as shown in Figs. 3 and 13.

The balconies built between the towers at the level of the first floor also provide shelter for vans waiting their turn to pass under the hoists in the towers or being loaded at additional loading stations provided by doors in the wall of the building and by openings through the floor of the balcony.

The cantilevers supporting the towers and balconies are illustrated in Fig. 14, and have a projection of 12 ft. Most

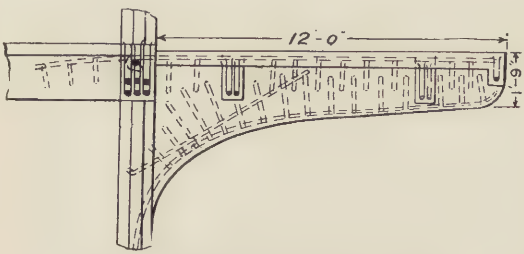


FIG. 14.—Section of a Balcony Cantilever.

of them are 20 in. deep at the free end, but the depth is varied according to the load to be carried, there being three standard types of construction for different parts of the buildings, the maximum depth being 2 ft. at the outer end and 6 ft. 6 in. at the supporting column.

As shown in Fig. 14, the reinforcement includes three systems of bars—one series at the top consisting of four horizontal bars, $1\frac{3}{4}$ in. diameter, these being continuations of the bars in the secondary beams of the first floor; another series at the bottom consisting of six $1\frac{3}{4}$-in. diameter bars, coming up from the wall and bent out so as to follow the curve of the cantilever; and one series consisting of $1\frac{3}{4}$-in. diameter bars, anchored into the wall and connecting the first two series in a direction tangential to the curve of the

cantilever close to the column. Two bars of the upper set are embedded for about 3 ft. into the beams of the first floor, and the other two are hooked round horizontal bars inserted in the wall columns. Two systems of stirrups are employed, in addition to the bars, for the purpose of resisting shearing stresses. The cantilevers are connected laterally by beams running parallel with the wall of the building, and over these is a floor slab of concrete-steel.

At the front of each shed a loading balcony, with a projection of 4 ft., is provided at the level of the first floor, being carried by concrete-steel cantilevers with a cast-iron nosing, so as to prevent injury by the accidental contact of goods being hoisted or lowered. Above this balcony a series of lifting platforms extends along the front of the sheds at the level of the second floor. These platforms, which are hinged so that they can be lifted up when not in use, are made of pitch pine with oak framing, and when lowered are supported by the iron brackets shown in Fig. 13. Similar hinged platforms are provided at each storey in front of the openings giving access to the hoisting towers.

15. Railway Tunnel through Shed.—A very interesting structural detail is the railway tunnel passing obliquely through the ground floor of one of the sheds, in order to permit goods trains to pass from the dock quay to the sidings. As illustrated in Fig. 15, this passage is completely isolated from the sheds by walls of concrete-steel to avoid all risk of fire. The arched beams carrying the floors above have a span of 50 ft., and support a superload of 260 tons each.

16. Precautions against Fire.—Being built entirely of concrete-steel, the sheds themselves are incombustible, but for the purpose of enabling the officials to isolate any local outbreak of fire in the goods stored, and to deal with it in the most efficient manner, the various floors of each shed are divided into two parts by a 4-in. concrete-steel party wall, extending from front to back. In each party wall there are two openings 12 ft. wide, fitted with double sliding doors, these and all other doors in the buildings being of fire-resisting construction.

Each shed is provided with three stairways, situated at

FIG. 15.—Railway Tunnel through a Shed.

each end and in the middle of the building, as indicated in Fig. 4. Each staircase includes forty-eight steps, twenty-four between the ground and first floors, twelve between the first and second floors, and twelve between the second floor and roof. The enclosures, treads, and all other details of these staircases are of concrete-steel.

17. Roof Details.—The roof parapets along the sides and back of each shed are formed by a continuation of the concrete-steel outer walls. These parapets are 39 in. high, and inside them runs a concrete-steel rain-water gutter. The parapet at the front of the sheds is of cast iron, secured by foundation bolts let into the concrete below, but the reason for the choice of this material is not very apparent. Concrete foundations have been built on the roof, between the bridge and the front parapet, for the 30-cwt. electrically driven hoists previously mentioned.

The floors and flat roof of each shed are finished with asphalt. On the ground floor there are two $\frac{3}{4}$-in. layers of this material laid in sheets with overlapping edges, on the first and second floors two $\frac{3}{4}$-in. layers, and on the roof two $\frac{1}{2}$-in. layers with overlapping edges.

18. Bridges between Sheds.—The bridges, or gangways, connecting the adjacent buildings are about 24 ft. long by 12 ft. wide. Each bridge is supported upon two rolled steel joists, 16 in. deep, connected by four $1\frac{3}{8}$-in. diameter tie-bars. The decking and other details of the bridges are composed entirely of reinforced concrete, and it should be noted that the concrete is quite separated from that of the sheds, gaps $1\frac{1}{2}$ in. wide being left at each end to provide space for expansion in hot weather.

19. Provisions for Mechanical Plant, Pipes, and Electric Cables.—One interesting feature in connection with the design and construction of these buildings was the very complete manner in which provision was made for embedding bolts, hooks, and other fastenings for mechanical appliances and electric cables, and for forming conduits to receive pipes and electric light wires. Provision for the numerous fastenings had to be made in the timber moulds for the beams and other members. In some places it was necessary to insert timber blocks, so as to form holes in the

concrete, wherein the fastenings could be caulked with cement, and in other places the fastenings themselves had to be inserted through the sides or bottom of the moulds. Owing to the great number of the different attachments, and the immense variety of size and shape, considerable care was necessary to avoid mistakes.

Provision for running electric-light wires was made by embedding tubes in the concrete near the upper surface of the floor beams. These tubes were filled with dry sand, so as to enable them to withstand the weight of the concrete, the ends being sealed up with plaster of Paris. After completion of the concreting the ends of the tubes were unsealed, and, the sand being removed, the conduits were ready for wiring.

The main cables for the supply of electricity for power and light are suspended from the under side of the floors by means of eye-hooks screwed into blocks of wood, which were set in the beams during construction, as described above.

20. Column and Floor Tests.—In concluding this description of the warehouses we cannot do better than give some results obtained during the tests which were conducted on the columns and floors in March 1904.

We will first take the test of a column on the second floor. The area covered by the load is indicated in Fig. 16, and the results of the tests are given in Table I. Fourteen instruments were employed for the measurement of deflection, the positions of these being shown by numerals on the plan.

All the floors and the flat roofs were calculated for a normal load of 1,875 kilogrammes per square metre (384.37 lb. per sq. ft.), and were tested to one and a half times this load—namely, to 2,800 kilogrammes per square metre (574 lb. per sq. ft.).

As representative examples of floors tests, we have selected three which were conducted upon main and secondary beams on the second storey. The areas covered by the loading are shown in Figs. 17, 18, and 19, and the results are given in Table II.

Tests Nos. 2, 3, and 4 were conducted thus:—

The area ABCD (Fig. 17) was first loaded up to 45 tonnes, then to 90 tonnes, and finally to the full test load of 410 tonnes.

The area EFGH (Fig. 18) was first loaded by the transfer of 45 tonnes from ABCD, then 45 tonnes and 50 tonnes were successively transferred from ABCD, making

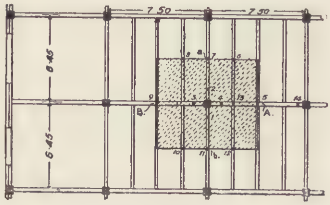


FIG. 16.

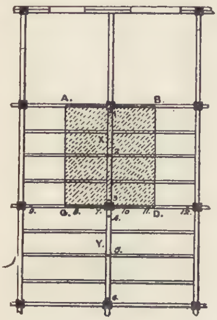


FIG. 17.

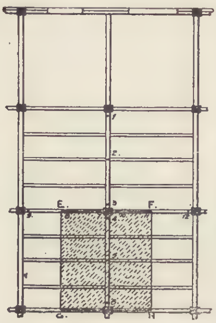


FIG. 18.

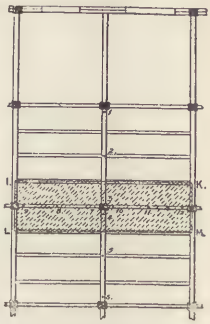


FIG. 19.

up the full test load of 140 tonnes. Thus, when the load of 45 tonnes was on EFGH, 95 tonnes still remained on ABCD, and when the load of 95 tonnes was on EFGH 45 tonnes remained on ABCD.

A similar method of procedure was followed in transferring the load from the area EFGH to the area IKLM (Fig. 19).

During the loading of ABCD the beam Y, which is a

continuation of the beam X, indicated an inverse deflection, being bent in an upward direction. The same effect was produced in X while Y was under load. It should be noticed that the floor areas tested exhibited remarkable elasticity.

The mean deflection shown by tests Nos. 2 and 3 measured only 8.7 millimetres and 8.6 millimetres respectively. As the permissible amount of deflection was 12.1 millimetres, these results are very satisfactory. The mean deflection observed in test No. 4 was 5.5 millimetres, the permanent set after unloading being inappreciable.

[Table I.

TABLE I.—TEST NO. 1, UPON A COLUMN ON THE SECOND STOREY OF SHED NO. 5 (see FIG. 16).
TEST LOAD, 140 TONNES.

All Readings are given in Millimetres.

Numbers of Instruments.	Readings taken during Loading.					Readings taken during Unloading.					
	Before Loading.	At 45 Tonnes.	At 90 Tonnes.	At 140 Tonnes.							
Hours.	Morning.					Afternoon.					
	8 h. 40 m.	9 h. 40 m.	10 h. 50 m.	11 h. 50 m.	2 h.	2 h. 15 m.	2 h. 30 m.	2 h. 45 m.	3 h.	3 h. 35 m.	4 h.
	mm.	mm.	mm.	mm.	mm.	mm.	mm.	mm.	mm.	mm.	mm.
1	2.0	2.2	2.5	3.0	3.0		Maximum Compression.				
2	2.0	2.2	2.3	2.6	2.6	..	..	1.4	..	..	..
3	2.0	2.4	2.9	3.3	3.3	3.3	3.0	2.7	2.3	1.5	1.6
4	2.0	2.3	2.8	3.4	3.4	Deflection of Main Beam (B)				.. ..	3.1
5	2.0	3.0	4.4	6.0	6.2	,,	,,	,,	(A)	.. ..	2.0
6	2.0	3.0	4.6	6.2	6.3	..	.	..	..	..	..
7	2.0	2.8	4.0	5.0	5.0	Deflection of Secondary Beam (*a*)				.. ..	2.4
9	2.0	3.2	4.3	5.3	5.4	5.4	4.4	3.4	2.0	0.5	0.5
11	2.0	2.8	3.6	4.9	4.9	Deflection of Secondary Beam (*b*)				.. ..	1.9
12	2.0	3.2	4.5	6.2	6.2	..	..	..	..	..	..
13	2.0	2.8	4.1	5.5	5.6	..	..	..	..	..	..
14	2.0	2.2	2.4	2.9	2.9	..	..	..	..	..	..

TABLE II.—FLOOR TESTS ON THE SECOND STOREY OF SHED NO. 5 (see FIGS. 17, 18, and 19).

All Readings are given in Millimetres.

—	Test No. 2, Fig. 17.						
Numbers of Instruments.	Readings from the Instruments.						
	Before Loading.	At 45 Tonnes.	At 90 Tonnes.		At 140 Tonnes.		Deflection.
Dates and Hours.	March 17.			March 18.			
	Morning.		After-noon.	Morning.			
	9 h.	12 h.	4 h. 30 m.	7 h.	9 h.	10 h. 30 m.	
	mm.	mm.	mm.	mm.	mm.	mm.	mm.
1	2.0	2.6	3.5	3.5	4.1	4.2	
2	2.0	4.5	8.5	8.9	12.7	12.9	8.7
3	2.0	2.6	3.3	3.3	4.1	4.2	
4	2.0	1.8	1.9	1.3	1.2	1.2	
5	2.0	1.3	0.7	0.6	0.2	0.2	
6	2.0	2.0	1.7	1.7	1.6	1.6	
7	2 0	1.9	1.9	1.9	1.9	1.9	
8	2.0	2.2	2.5	2.5	2.8	2.8	
9	2.0	1.9	1.9	1.9	1.9	1.9	
10	5.0	3.8	3.3	3.2	3.2	3.2	
11	2.0	2.3	2.8	2.8	3.3	3.4	
12	2.0	2.1	2.0	1.9	1.9	1.9	

—	Test No. 3, Fig. 18.				
Numbers of Instruments.	Readings from the Instruments.				
	At 45 Tonnes.	At 90 Tonnes.	At 140 Tonnes.		Deflection.
Dates and Hours.	March 18.				
	Morning.	Afternoon.			
	11 h. 40 m.	1 h. 45 m.	3 h. 5 m.	4 h. 5 m.	
	mm.	mm.	mm.	mm.	mm.
1	3.6	3.0	2.2	2.2	
2	9.9	6.3	3.0	2.8	
3	3.4	2.5	1.6	1.6	
4	2.0	3.2	4.3	4.5	
5	3.4	8.0	12.6	13.0	8.6
6	2.4	3.4	4.3	4.4	
7	1.9	1.4	1.8	1.8	
8	2.7	2.5	2.5	2.4	
9	1.9	1.9	2.0	2.0	
10	2.0	1.8	1.8	1.8	
11	3.4	3.4	3.8	3.8	
12	1.9	1.9	2.0	1.9	

—	Test No. 4, Fig. 19.							
Numbers of Instruments.	Readings from the Instruments.							
	At 45 Tonnes.	At 90 Tonnes.	At 140 Tonnes.		Normal Load.	After Unloading.		Deflection.
Dates and Hours.	March 19.							
	Morning.					Afternoon.		
	7 h.	8 h.	9 h.	10 h.	11 h.	12 h. 30 m.	10 h.	
	mm.	mm.	mm.	mm.	mm.	mm.	mm.	mm.
1	2.2	2.2	2.4	2.4	2.3	2.0	2.0	
2	3.8	3.9	5.0	5.0	4.5	3.5	3.2	
3	2.3	2.6	3.1	3.1	3.0	2.4	2.2	
4	3.7	3.6	3.2	3.2	3.2	2.6	2.4	
5	9.1	8.6	5.6	5.6	5.2	3.9	3.2	
6	3.7	3.5	2.8	2.8	2.7	2.6	2.9	
7	1.7	1.9	1.9	1.9	1.8	1.9	1.9	
8	4.0	4.2	5.6	5.7	5.2	2.3	2.0	
9	2.1	2.1	2.3	2.3	2.2	1.8	2.0	
10	1.2	1.2	1.3	1.3	1.2	0.6	0.4	5.5
11	4.9	5.3	7.5	7.7	7.2	3.2	3.0	
12	3.1	2.2	2.4	2.4	2.3	2.0	1.9	

CHAPTER II

STOREHOUSES AT THE GENNEVILLIERS GASWORKS, PARIS

Near the bank of the Seine on the northern boundary of the plain of Gennevilliers a large establishment has recently been built by the Société d'Eclairage, Chauffage, et Force-Motrice de Gennevilliers, including gas-producing plant, gasometres, storehouses, a coal wharf on the river front, and a concrete-steel viaduct carrying railway lines for the transportation of coal to the storage depôts.

21. General Description.—The construction of two large storehouses in the works forms the subject of this chapter. These buildings, with the exception of the outer brick walls, were built entirely of concrete-steel on the Coignet system by M. Stinville, the engineer by whom the entire establishment was designed. A photograph of the storehouses is reproduced in Fig. 20, but as the two buildings are identical in dimensions and construction it is only necessary to refer to one of them in the following description.

Each building is 45 metres long by 28 metres wide, comprising two floors and a flat roof, the height to the top surface of the latter being 18.30 metres.

Fig. 20 is an elevation of the principal façade of the storehouse. All the concrete-steel columns there shown project as pilasters, the concrete-steel main beams of the first and second floors and of the roof are also slightly in advance of the concrete-steel framing between the principal members, this framing consisting of stanchions and horizontal beams, each 15 centimetres square, flush with the panels or curtain walls of brick.

The elevation of the back and ends of the storehouse is of similar character, although the details of the arrangement

FIG. 20.—Gennevilliers Gasworks. View of Storehouses.

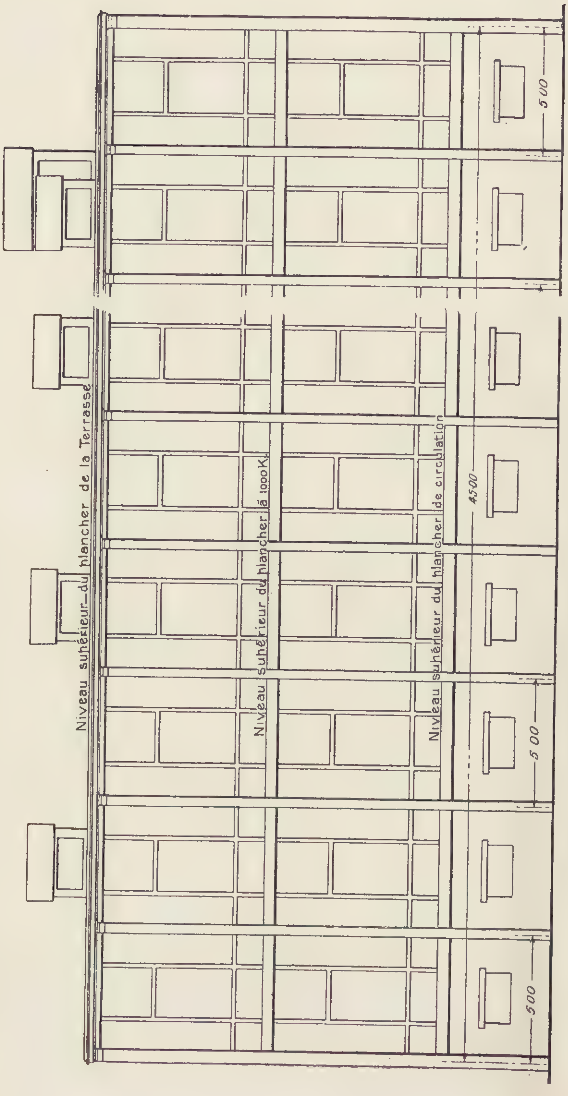


FIG. 21.—Gennevilliers Gasworks. Front Elevation of one Storehouse.

necessarily vary somewhat. As usual in concrete-steel construction of good design, the whole of the concrete members are monolithic, and all the reinforcement is securely anchored. Thus the framework of the building is a complete structure in itself, very much akin to the steel cage familiar to American architects, and although the outer walls are of brick a very large proportion of the total weight of the structure is carried by concrete-steel columns, and transmitted by them directly to the foundations.

The approximate thickness of the brickwork in the wall panels is 22 centimetres up to the level of the second floor, and 17 centimetres up to the roof.

Two outside stairways, each of twenty-five steps, lead from ground level up to a landing at first floor level, and inside stairways run from the first floor to the second floor and thence to the terrace roof.

22. Column Arrangement and Dimensions.—Fig. 22 is a half foundation plan showing the arrangement of the columns, which are of five different types, lettered G, H, I, K, L, for the purpose of identification. Each column is supported on a cast-iron base plate, and this in turn by a concrete pier of cylindrical form carried down to a sufficient depth into the earth to ensure stability, the top surface of the pier being 50 centimetres below ground level.

There are four columns of type K, one at each corner of the building, and for this type the concrete pier has the diameter of 2 metres. The cast-iron base measures 1.05 metre square at the bottom, 50 centimetres square at the top, and 40 centimetres high. The base is stiffened by eight ribs rising from the bottom plate to the top, where suitable provision is made for supporting the vertical reinforcement of the column.

All the remaining columns along the front and back walls of the buildings are of type H, there being eight of these between the corner columns at the end of each façade, or including the latter ten columns in each wall.

The foundation piers are of the same diameter as for columns of type K, namely, 2 metres. The cast-iron bases also measure 1.05 metre square at the bottom, but at the top the dimensions are 50 centimetres by 35 centi-

metres, to correspond with the transverse dimensions of the columns.

Along each end wall of the warehouse there are, in addition to the two corner columns of type K, six columns of type I, measuring 50 centimetres by 35 centimetres, making eight columns in each wall. The cylinder founda-

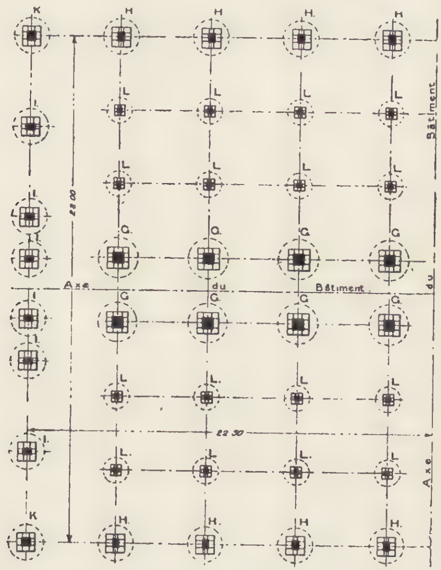


FIG. 22.—Half Foundation Plan.

tions and the cast-iron bases for the type I columns are of the same dimensions as for the columns of types H and K.

On either side of the longitudinal axis of the building there is a row of eight columns, type G, in addition to two of type I, making ten columns in each row. The foundation piers for these interior columns are of 2 metres diameter; the cast-iron bases measure 1.20 metres square at the bottom, and 60 centimetres square at the top, the

columns themselves having a sectional area of 60 centi-metres square.

The other interior columns are those of type L, which are of much smaller dimensions, being intended merely for the support of a load below the level of the first floor. The cylinder piers for these columns are 1.40 metres in diameter, the cast-iron bases are 80 centimetres square at

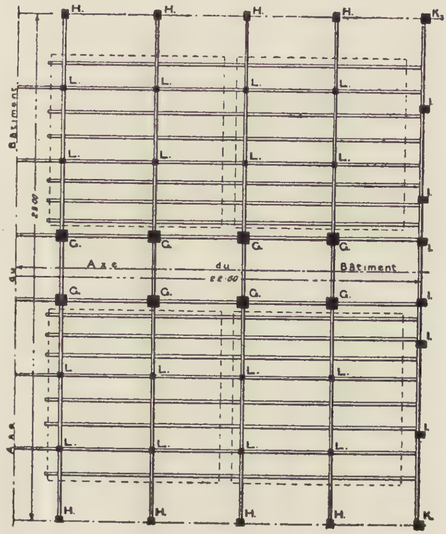


FIG. 23.—Half Plan of Tank Beam System.

the bottom, while the columns themselves measure 25 centimetres square.

The columns generally are disposed in rows 5 metres apart, centre to centre, measured along the main axis of the building, but the spacing of the columns in the transverse rows is not uniform. At each end wall two of the I type columns are spaced 3.35 metres apart, one on either side of the centre line, then comes another column at a distance of 2.325 metres, and finally come two outer columns at intervals of 5 metres apart.

The columns in each of the other rows are spaced on either side of the centre line as follow :—

Centre to column G	1.80 metres.
Column G to column I . . .	4.10 ,,
Column I to column L . . .	4.05 ,,
Column L to column H . . .	4.05 ,,

The dimensions given for the main columns in the walls of the building, and for the interior columns, only apply so far as concerns those portions of their length extending up to the beams supporting the purification tanks and first floor (see Articles 23 and 24). Above that height the G and I type columns are reduced in sectional area proportionately with the reduced load to be carried, and none of the L type columns are continued up to first floor level, since the only reason for their employment was to afford intermediate support for the members beneath the tanks.

23. Purification Tank Supports.—The reason for the arrangement of the columns in the manner indicated is to provide for supporting eight purification tanks, each measuring 9.44 metres square by 1.50 metre deep, and weighing 140 tons, including the contents. These tanks are carried upon a system of main and secondary beams, the ends of the main beams being supported by the outer columns of types H, I, and K in the front, back, and end walls, and by the columns of type G on either side of the centre line of the building, while intermediate support is afforded by the columns of type L.

Fig. 23 is a half plan of the tank beam system, which, as explained in Article 24, does not constitute any part of the first floor proper.

Referring to this drawing, the main beams extending from front to back of the building are supported by rows of columns, and the spacing between each beam is consequently 5.00 metres, centre to centre. All these beams are of concrete-steel, and measure 16 centimetres wide by 45 centimetres deep.

Below the areas covered by the tanks, and along the lines of columns G, the main beams are connected by secondary beams with a cross section measuring 16 centimetres wide by 40 centimetres deep.

On either side of the longitudinal axis of the building the secondary beams are spaced as follow :—

From axis to line of columns G . .	1.80 metres.
From line G to first beam	0.80 ,,
Between the next three beams . .	1.10 ,,
Between the next four beams . . .	1.35 ,,
From last beam to centre of columns H	2.70 ,,

The positions of the four tanks in one-half of the building are indicated in Fig. 23 by broken lines. Inspection will show that there are ample spaces between the tanks themselves, as well as between them and the ends and sides of the building. The superload on the tank beam system was specified at 1,500 kilogrammes per square metre, about 307 lb. per square foot.

24. First Floor Construction.—The spaces around the tanks are occupied, as shown in Figs. 24 and 25, by the first floor, which consists of beams and a floor slab supported along the outer edges by the columns H, K, and I, and elsewhere by dwarf columns built up from the beams upon which the tanks are placed. In this plan the outer brick walls are indicated, together with the 15-centimetre square concrete-steel stanchions, which, together with the main columns and horizontal members in this and the upper storey, constitute a complete skeleton framing for the entire building.

For supporting the floor, beams 16 centimetres wide by 42 centimetres deep are carried along the front, back, and end walls between the columns. Then, at a distance of 1.03 metres from these beams, there is a line of dwarf columns, with the cross section of 15 centimetres square, built up from the secondary beams below the tanks. These four rows are situated one near each end and side wall of the building, and one at each side of the longitudinal centre line.

Upon the dwarf columns are beams 25 centimetres wide by 17 centimetres deep, for a concrete-steel floor slab, 6 centimetres thick, which extends over the two parallel series of beams, forming a continuous platform round the building.

The front or inner edge of the platform is rebated for

the addition of wood flooring up to the tanks. The three transverse gangways between the tanks, two 65 centimetres

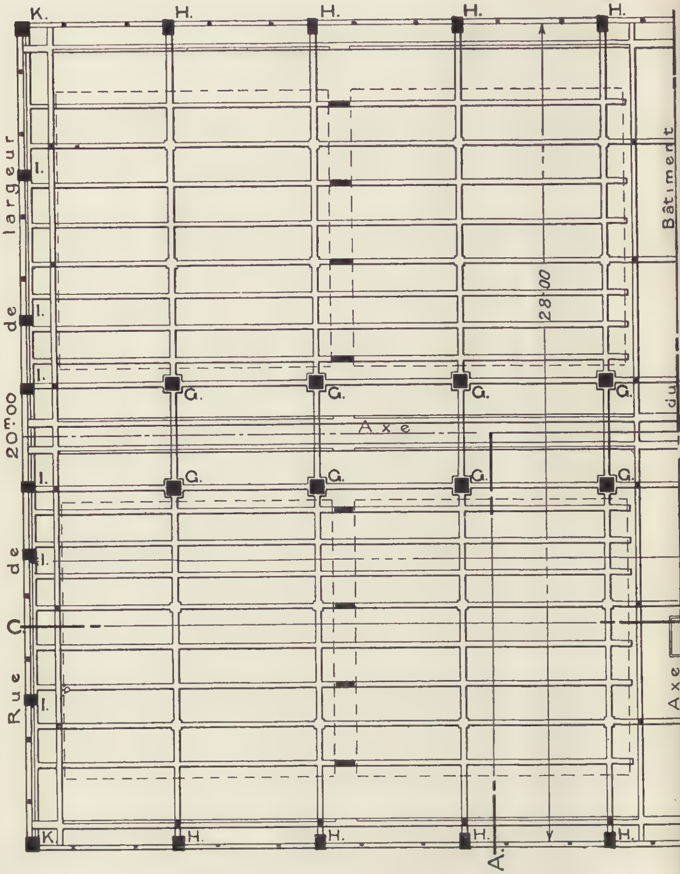


FIG. 24.—Plan of First Floor. (Left-hand half.)

wide and one 3.50 metres wide, consist of concrete-steel floor slabs 12 centimetres thick, with suitable support from

below, but the longitudinal gangway extending from end to end of the building is floored with concrete-steel only in

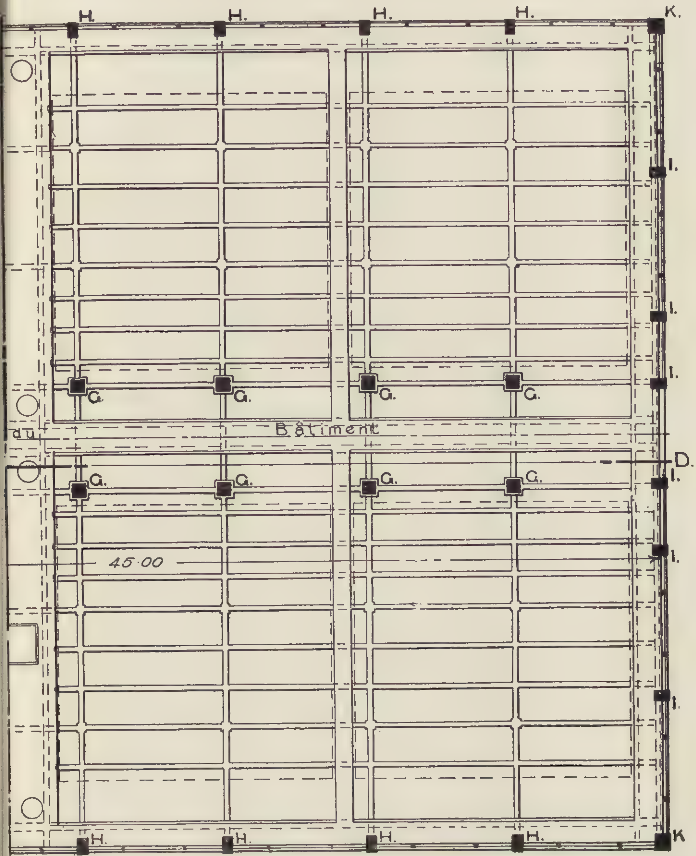


FIG. 25.—Plan of First Floor. (Right-hand half.)

the centre for a width of 1 metre, the remainder of the floor up to the tanks on either side being of timber.

Fig. 26 is a part cross section of the building, and Fig. 27 a half-longitudinal section. These drawings will serve the purpose of further explaining the details of construction.

From the particulars stated above it will be seen that the first floor is really a platform pierced by eight openings, in each of which is a large shallow tank, partially sunk below

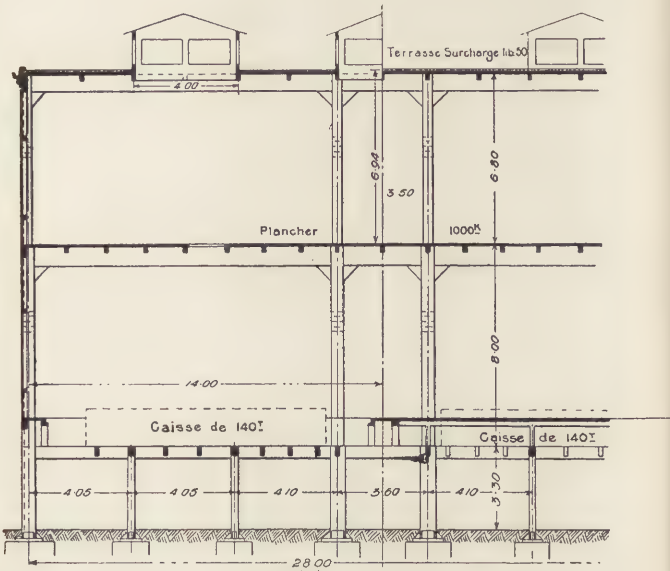


FIG. 26.—Part Cross Section of Storehouse.

the floor level, above which the upper part of each tank projects to a height of 40 centimetres. The calculated superload on this floor was 500 kilogrammes per square metre, about 102.5 lb. per square foot.

25. Second Floor Construction. — To show the general arrangement of the main and secondary beams in the second floor, we give in Fig. 28 a half plan of the storehouse at that level.

The dimensions of the columns were stated in Article 22, and, with the exception of type G columns, the sectional areas are the same as in the lower portion of the building. Along the front and back walls the columns K and H are connected by wall beams measuring 16 centimetres wide by 40 centimetres deep, these members being

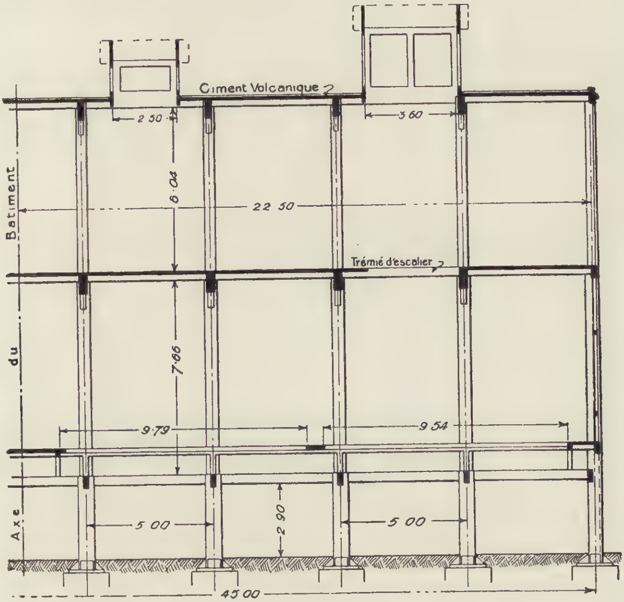


FIG. 27.—Half Longitudinal Section of Storehouse.

monolithic with the columns, and receiving intermediate support from the small 15-centimetre square stanchions between the columns in the storey below. The beams at each end wall are 24 centimetres wide by 40 centimetres deep; these also have intermediate support from 15-centimetre square stanchions, except in the three centre spans.

The main beams perpendicular to the longitudinal axis of the building are 30 centimetres wide by 78 centimetres

deep, those at the front and back having the clear span of 11.28 metres, while between the longitudinal rows of columns G the clear span is 3 metres. All the secondary beams have the dimensions of 16 centimetres wide by 26 centimetres deep.

Over the whole beam system a concrete-steel floor slab is

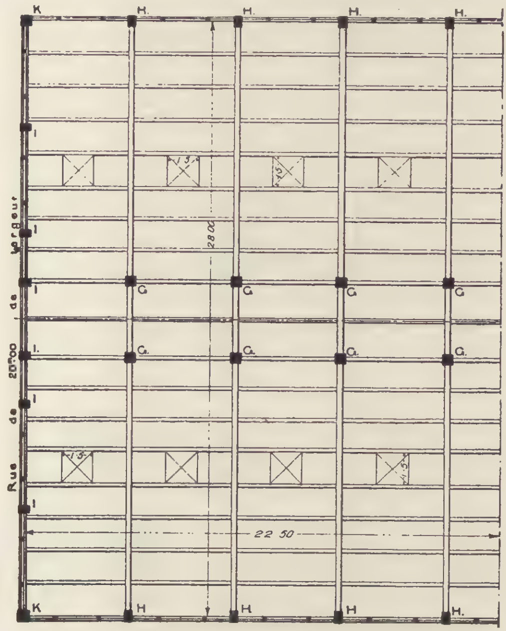


FIG. 28.—Half Second Floor Plan of Storehouse.

formed, in which two openings, 1.50 metre square, are left over each tank on the floor below. Thus there are sixteen openings in all. The thickness of the floor slab is 8 centimetres, but along the gangways, or passages running from end to end and from side to side of the building the thickness is increased to 9 centimetres. The stairways leading up from the storey below and to the terrace roof

above this floor are situated in the longitudinal passage, but are not shown in Fig. 28, as they occur in the opposite half of the building.

The second floor was designed for a superload of 1,000 kilogrammes per square metre, about 205 lb. per square foot.

26. Details of Reinforcement.—Figs. 29 and 30 contain details of a type H column, and the main and secondary beams connected therewith, and Figs. 31 and 32 give similar details of the construction in connection with a column of type G.

In the lower drawing of Fig. 29 we have a horizontal section showing the vertical reinforcement in the column H, consisting of four steel bars of 42 millimetres diameter, one at each corner and two bars of 20 millimetres diameter, all six bars being placed within about 20 millimetres of the outer surface of the concrete, and tied by transverse spiral hooping of 6-millimetre diameter wire. The same drawing shows small portions of two 16-centimetre by 40-centimetre wall beams FF, and part of one 30-centimetre by 78-centimetre main beam A. The reinforcing bars of these members meet in the column, and are securely connected by the concrete.

Further details of the column and beam construction are given in the upper drawing of Fig. 29 and in Fig. 30, where the spiral reinforcement of the column H is clearly indicated. The longitudinal reinforcement of the wall beams consists of two bars each of 18 millimetres diameter situated in the floor slab. Fig. 30 shows that the four bars of longitudinal reinforcement in the wall beam are connected by vertical loops of steel wire, for withstanding shearing stresses, and that over the two bars, which are of 6 millimetres diameter, a thin rod is placed, this being for the equalisation of stress. Similar vertical loops and transverse rods are placed at frequent intervals in the length of each beam.

The longitudinal reinforcement of the main beam includes eight bars of 41 millimetres diameter in the tension area, in two rows of four bars, between them being a short bar of 14 millimetres diameter to distribute the stress. Similar pieces are placed at intervals along the beam. There are

four bars of 43 millimetres diameter in the compression area, and the two series are connected, as in the case of all other beams, by vertical ties of 6 millimetres diameter. Similar ties, but of reduced diameter, are employed at

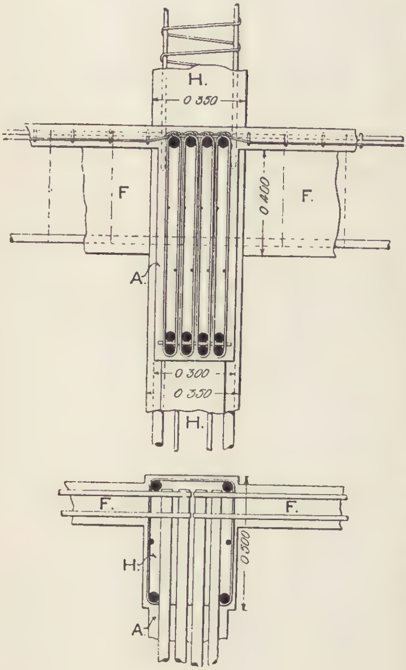


FIG. 29.

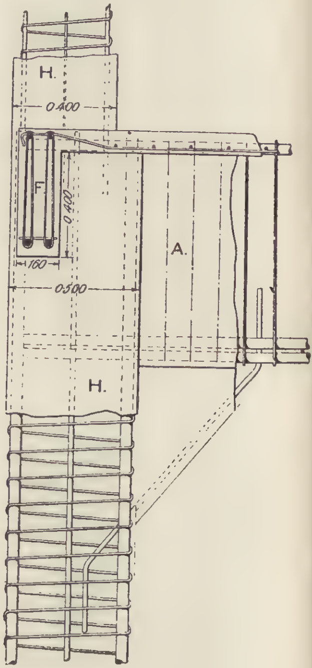


FIG. 30.

intervals of about 8 centimetres apart, as indicated by broken lines in Fig. 30.

The upper portion of each column is extended to form brackets with the object of affording more rigid support to the main beams.

Fig. 30 illustrates the reinforcement applied to these

brackets. It simply consists of two bars of 16 millimetres diameter bent at each end so as to provide for secure anchorage into the concrete of the columns and floor beams respectively, the brackets themselves measuring 60 centimetres high by 50 centimetres wide.

The reinforcement of the floor slabs between the main and secondary beams consists of 6-millimetre and 8-millimetre diameter rods disposed crosswise so as to form a network with meshes from 13 to 14 centimetres square. It will be seen by the drawings that all these rods are securely connected with the reinforcement of the beams in order to bind the construction together.

Above the level of the second floor the dimensions of the H type columns are reduced to 40 centimetres by 35 centimetres, and the four corner verticals to 16 millimetres in place of 42 millimetres in the storey below, and the two intermediate bars of 20 millimetres diameter are replaced by others of 10 millimetres diameter. The spiral hooping, however, still consists of 6-millimetre diameter wire.

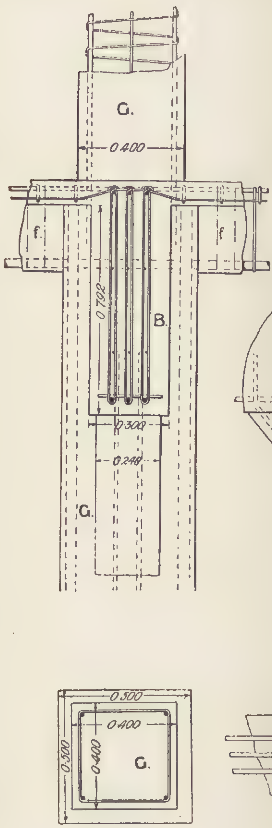
Figs. 31 and 32 contain drawings illustrating the details of type G columns, and the floor system in connection therewith. The general construction is very much like that already described, but as the span of the main beams is comparatively small the proportion of steel is very much less than in the case of the beams proceeding from the type H columns.

Having no wall load to carry, the corner vertical bars of the type G columns have the diameter of 36 millimetres in place of 42 millimetres, and there are four 10-millimetre bars, one in the middle of each side. The spiral hooping, as before, is of 6 millimetres diameter.

Columns of type G are reduced in area above the level of the second floor to 40 centimetres square, and the reinforcement is reduced to the dimensions of 16 millimetres diameter for the four corner bars, while the four intermediate bars are kept at the diameter of 10 millimetres. The cross section at the bottom of Fig. 31 indicates the comparative dimensions of the upper and lower sections of a type G column.

The cross section at the bottom of Fig. 32 represents the

secure manner in which the horizontal reinforcement of two



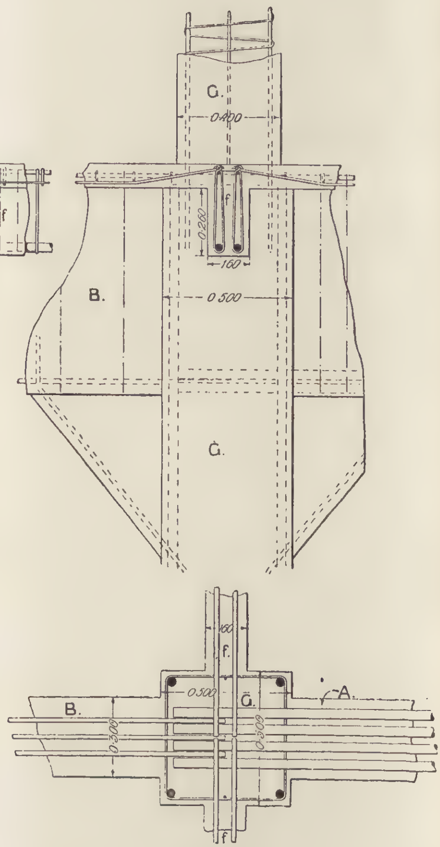


FIG. 31. FIG. 32.

main beams A and B is interlocked at the column. Owing to the shorter span of beam B in this case only three bars

are required in the tension area, these bars being of 18 millimetres diameter, and over them are distributing rods as in all the other beams, the diameter in this case being 10 millimetres. The three bars in the compression area of the same beam are of 14 millimetres diameter, and they are connected with the lower series by vertical ties of 4 millimetres diameter, these ties occurring at intervals of 10 centimetres along the beam.

27. Roof Construction.—Fig. 33 is a half-plan of the terrace roof showing the arrangement of the beams and the position of the lanterns.

The type I columns are reduced above the level of the second floor to 40 centimetres by 35 centimetres, with a corresponding reduction of the reinforcement, but the type K columns are continued without alteration of transverse dimensions to the top of the building.

The roof beams at the front and back of the building are 16 centimetres deep, and those along the ends are of the same dimensions. These beams receive intermediate support from 15-centimetre square stanchions in the walls, as in the case of the first and second floors. The main beams running from front to back are 24 centimetres wide by 70 centimetres deep, and, like those on the floor below, receive additional support at the columns from bracketed projections on the last-mentioned members. The secondary beams measure 10 centimetres wide by 25 centimetres deep, and are spaced 2.025 metres apart, with the exception of two 1.80-metre spans between the type G columns.

As the roof has only to bear its own weight, the terrace was calculated for the very small superload of 50 kilogrammes per square metre (10.25 lb. per square foot). Consequently, the dimensions of the beams are much smaller than those on the second floor, the difference being much more marked in the secondary than in the main beams.

The roof slab is of reinforced concrete 6 centimetres thick, and has a fall of 1 centimetre per metre to provide for the flow of rain-water into the gutters. The slab is pierced by twelve openings for the lantern structures, these openings being bordered by trimmers of the same width and depth as the secondary beams, and continued

up with a thickness of 8 centimetres to form the walls of the lanterns. The eight lanterns at the front and back of the building have the uniform dimensions of 4.00 metres by 2.50 metres in plan, and of those in the centre row

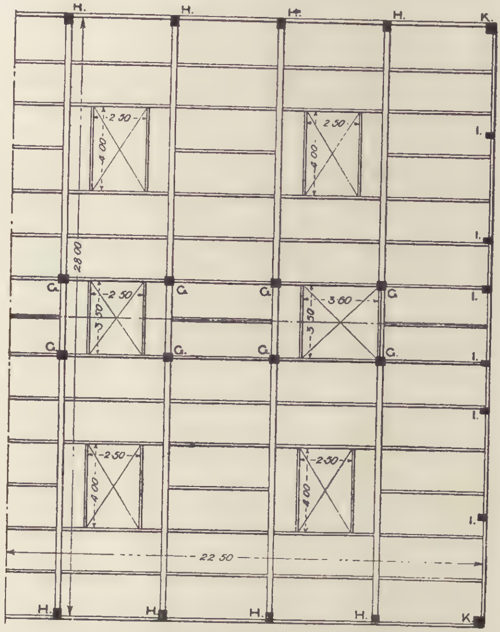


FIG. 33.—Half Roof Plan of Storehouse.

three measure 3.50 metres by 2.50 metres, and one, which covers the staircase enclosure, 3.50 metres by 3.60 metres.

Over the concrete of the roof slab is a layer of rock asphalt to avoid the risk of percolation, and fine gravel is spread upon the asphalt to the depth of 1 centimetre, as a protection against the direct rays of the sun.

CHAPTER III

The Great Western Railway Stationery Warehouse, London—A Chicago Warehouse Building

THE GREAT WESTERN RAILWAY STATIONERY WAREHOUSE, LONDON

28. General Design.—The building recently completed by the Great Western Railway Company, at Royal Oak, London, for the storage of stationery and kindred purposes is an example of framed construction, and so far as principles of design are concerned is similar to American buildings having a complete steel skeleton capable of carrying all floor loads and transmitting them through columns to predetermined points on the site.

In the case here considered the framework consists entirely of reinforced concrete on the Hennebique system. It includes columns, girders, joists, floor slabs, and a flat roof, constituting a monolithic structure of immense strength and rigidity. The outer walls are of red brick, supported in great measure by the reinforced concrete frame. The general design is illustrated by Fig. 34, which is a longitudinal section, the front of the warehouse being at the left-hand side of the drawing.

The building measures 104 ft. 6 in. long by 80 ft. 9 in. wide by 68 ft. high from ground level to the top of the parapet. It includes a basement, four upper storeys, and a terrace roof.

The basement extends beneath the footpath in front, the ground being held up by a reinforced concrete retaining wall 6 in. thick, with counterforts 12 in. square at intervals of 5 ft. apart, centre to centre, as illustrated in Figs. 35 and 36.

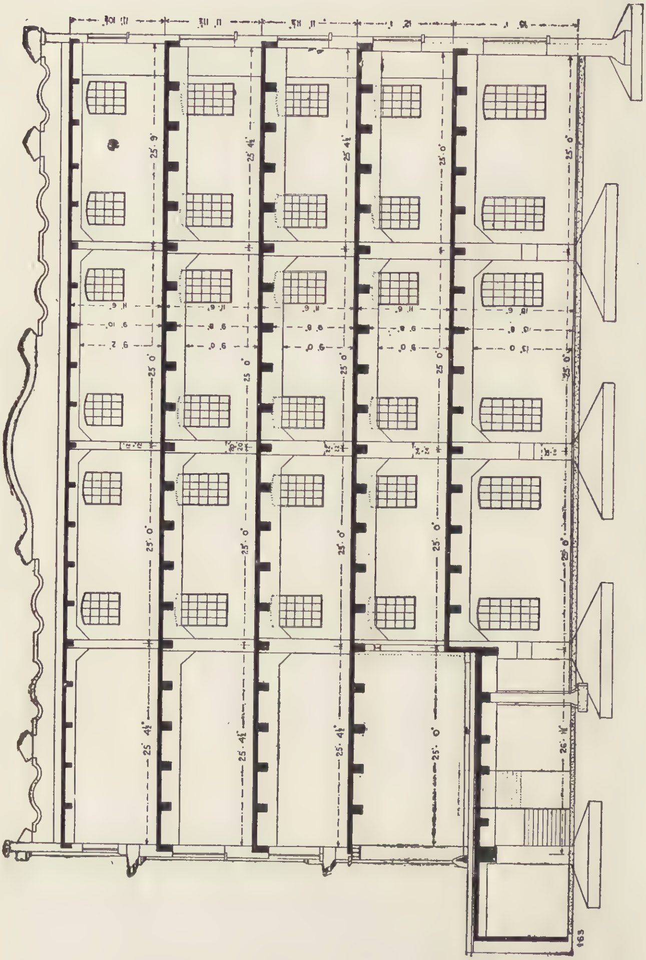


FIG. 34.—Great Western Railway Stationery Warehouse (Section).

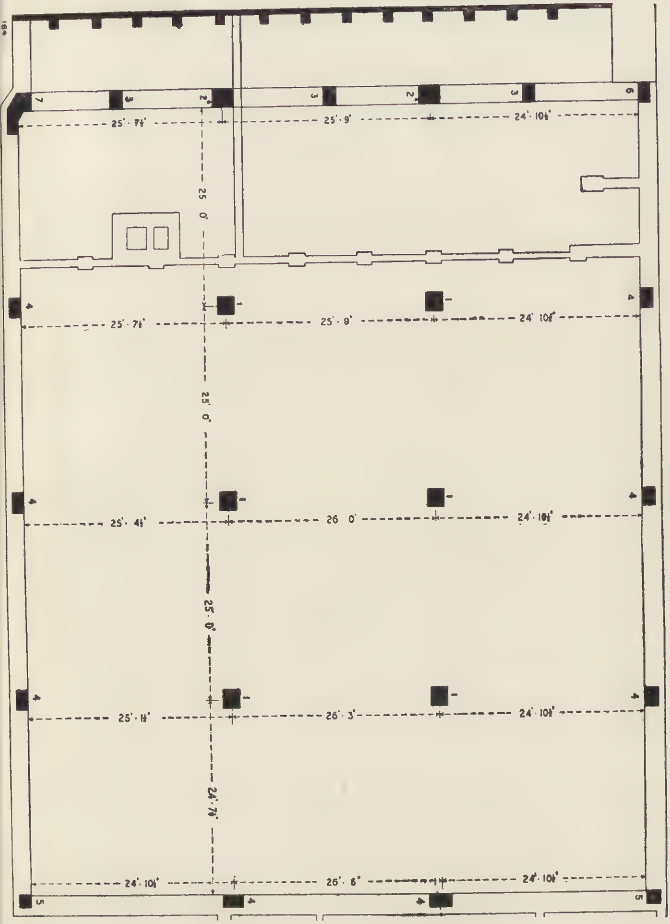


FIG. 35.—Plan.

The various floors were designed to afford the accommodation stated below:—

Basement—	Accumulator room for electric light; Boiler house for heating apparatus; Packing department.
Ground Floor—	Offices and stationery stores.
First Floor—	Printing and bookbinding departments.
Second Floor—	Stores for books and papers.
Third Floor—	Stores for books and papers.
Roof—	Storage of materials not liable to injury by weather.

It is stated by Mr. W. Armstrong, M.Inst.C.E.,[1] who designed the warehouse for the Great Western Railway, that the chief reasons which influenced the company in favour of reinforced concrete construction were the relative economy and rapidity of erection as compared with ordinary building construction. But another reason was found in the capacity of reinforced concrete to carry the heavy loads demanded, and to withstand the severe stresses due to the constant vibration anticipated from the running of fast trains within 5 feet of the building. It may be assumed that the valuable fire-resisting properties of the construction also received due consideration.

29. Foundations.—The foundations are in yellow clay, the column bases and other footings being extended so as to keep the pressure on the ground within the limit of 3 tons per square foot. Fig. 35 is a plan showing the position of the reinforced concrete columns, and reference to Fig. 34 will enable the reader to form an idea of the column bases.

30. Column Construction.—The following are the transverse dimensions of the various columns up to ground floor level:—

Type 1	2 ft. 2 in. by 2 ft. 2 in.
,, 2A	2 ft. 4 in. by 2 ft. 4 in.
,, 2B	3 ft. 1 in. by 2 ft. 4 in.
,, 3	2 ft. 4 in. by 1 ft. 2 in.
,, 4	2 ft. 4 in. by 1 ft. 6 in.
,, 5	1 ft. 6 in. by 1 ft. 6 in.
,, 6	2 ft. 4 in. by 1 ft. 6 in.
,, 7	(sectional area $10\frac{1}{2}$ sq. ft.)

[1] *Concrete and Constructional Engineering*, Vol. I. No. 2.

Fig. 35 shows that twenty-three columns are to be found in the basement storey, but of these only twenty are carried to the top of the building. The other three (type 3) are merely intended as intermediate supports for the main beam beneath the front wall.

31. Column Reinforcement.—The number and diameter of the steel bars employed as reinforcement in the columns necessarily varies according to the shape of the columns and the load to be carried.

Fig. 36 is a section applying particularly to columns 2A and 2B, but may be taken as generally representing the

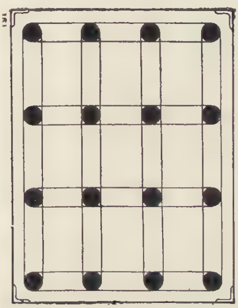


FIG. 36.—Horizontal Section of Main Column.

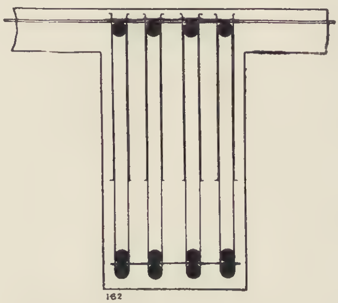


FIG. 37.—Cross Section of Main Beam.

system of reinforcement in the other columns. It should be noticed that the corners of the columns are protected against accidental injury by angle bars, which reach to the height of about 6 ft. above floor level.

The method of moulding the columns was substantially the same as that followed in the case of the Transit Sheds at Manchester Docks (see Article 7).

32. Floor and Roof Beams and Slabs.—The main beams, extending longitudinally through the building, are spaced at intervals which vary slightly with the spacing of the main columns, but may be averaged approximately at 25 ft. 6 in. from centre to centre. The exact intervals are figured in Fig. 35. The columns and main beams are

connected transversely by secondary beams spaced 5 ft. apart, centre to centre, thus dividing the floor system into panels measuring 25 ft by 5 ft.

Fig. 37 is a typical cross section showing the reinforcement of the main beams, which, with the exception of those in the flat roof, measure 18 in. wide by $35\frac{1}{2}$ in. deep, including the thickness of the floor slab.

Fig. 38 is a longitudinal section illustrating the construction of a main beam, the column connections, and the proportions of the secondary beams connecting the interior and wall columns.

The secondary beams are similar in construction to the main beams. Their general dimensions are 12 in. wide by $27\frac{1}{2}$ in. deep.

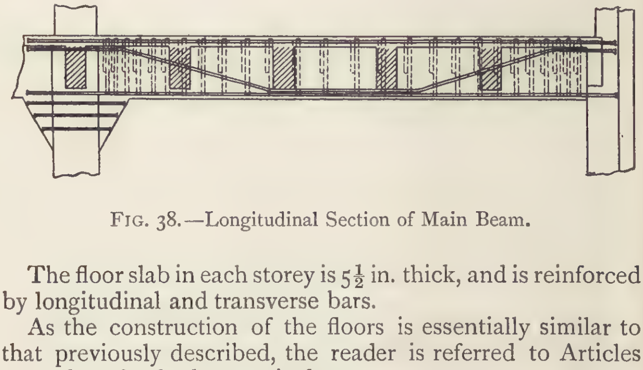


FIG. 38.—Longitudinal Section of Main Beam.

The floor slab in each storey is $5\frac{1}{2}$ in. thick, and is reinforced by longitudinal and transverse bars.

As the construction of the floors is essentially similar to that previously described, the reader is referred to Articles 11 and 12 for further particulars.

All the floors are designed for a superload of 5 cwt. per square foot, and according to the engineer's specification this load is to be carried without permanent deflection. Fig. 39 is a view which gives a good idea of the floor construction.

The roof is similar to the floors, but of far lighter construction, being proportioned for a superload of only about 62 lb. per square foot.

33. Concrete.—Thames ballast broken to the maximum size of $\frac{3}{4}$ in. formed the aggregate for the concrete used, the

proportions in general being 1 part of Portland cement to 5 parts of ballast. The average thickness of concrete outside the reinforcement was $1\frac{1}{2}$ in. for columns and 1 in. for the under side of the floors.

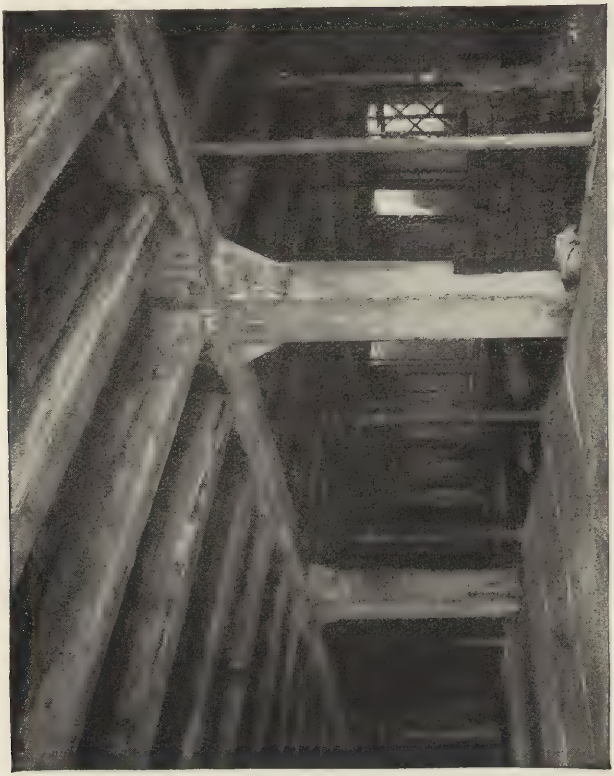


Fig. 39.—Great Western Railway Stationery Warehouse (Interior).

A CHICAGO WAREHOUSE BUILDING

34. Main Structural Features.—In this building the application of concrete-steel is limited to the interior, as

the outer walls are of brick masonry built in the ordinary manner. In accordance with the practice general in the United States, the main features of the building were designed by an architect, and the purely structural work was executed from the drawings and under the superintendence of a civil engineer.

The building, occupying a site measuring 100 ft. wide by 130 ft. deep, with frontages on Michigan Avenue and 13th Street, Chicago, was designed for seven storeys, of which four were finished and occupied in the year 1904, leaving the other three to be added as tenants offered themselves.

Fig. 40 includes a part sectional elevation through the outer wall beams and columns, and an elevation of one series of columns showing details of the beams and floor slabs.

The height of the different storeys are — ground floor, 10 ft. 6 in.; first storey, 15 ft.; second and third storeys, 12 ft. each, the clear height in each case being 1 ft. 6 in. less.

All the concrete-steel work, including column bases, columns, beams, joists, and floor slabs, is monolithic, being formed of Portland cement concrete of somewhat different proportions, reinforced by round rods of mild steel with an elastic limit of 36,000 lb. per square inch, these rods being of different diameter according to requirements.

35. Column Bases.—The columns are spaced 15 ft. 6 in. apart in rows running from front to back of the building, the rows being at a distance of 20 ft. one from another, measurements being taken from centre to centre in each case.

The column bases rest upon a layer of fine sand, permeated with water, above a stratum of blue clay, and the bases are proportioned so as to keep the load down to 4000 lb. per square foot.

Figs. 41 and 42 show the details of a typical column base, the area in this instance being 11 ft by 11 ft., but in some cases the dimensions are slightly less. No vertical bars or stirrups for resisting shear are employed, but the downward

thrust of the column is distributed by means of a horizontal plate of metal as shown in the section.

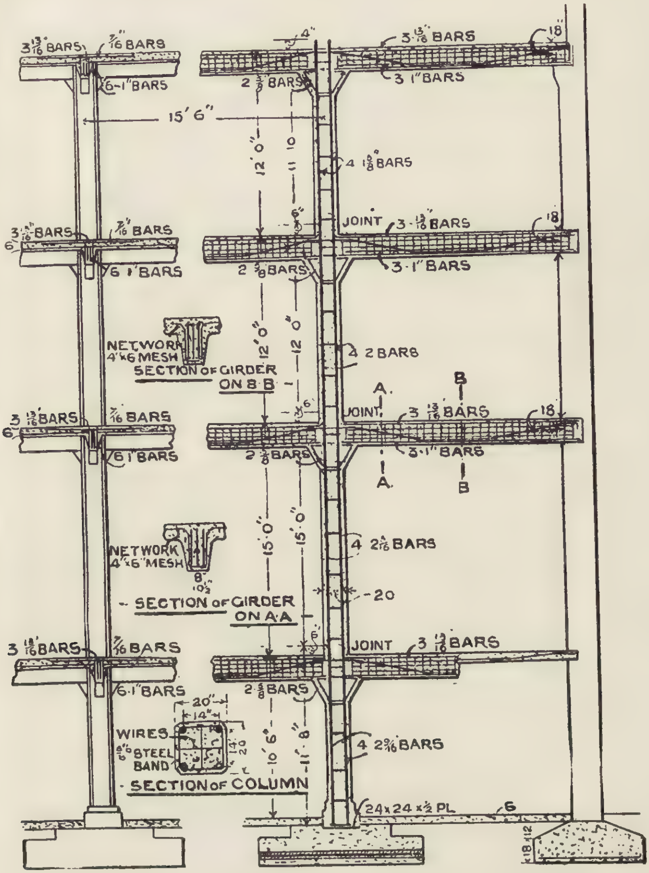


FIG. 40.—A Chicago Warehouse (Sectional Elevations).

In constructing the bases, a 4-in. layer of concrete was first formed, the proportions being 1 part of Portland cement,

2 parts of sand, and 3 parts of limestone crushed so as to pass through a $1\frac{1}{2}$ in. diameter ring. Four $\frac{1}{2}$ in. steel bars were then placed side by side near and parallel to each edge of the concrete slab, and over these two sets of 1 in. bars were laid nearer the centre, as shown in the plan, diagonally from corner to corner of the slab, and finally two sets of seven $1\frac{7}{8}$ in. bars were laid near the centre and parallel to the edges of the slab. All the bars were connected by means of two strands of 18 gauge wire so as to keep the bars in their correct positions. Cement mortar in the proportions of 1 : 4 was then poured upon and around the bars to form a layer 5 in. thick. The bars were shaken to permit the material to settle properly, and the concrete was thoroughly tamped.

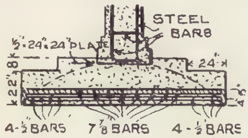


FIG. 41.

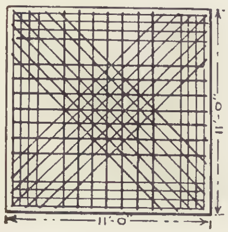


FIG. 42.

A layer of 1 : 2 : 3 limestone concrete was deposited so as to complete the footing to the dimensions and shape shown by the section in Fig. 41. The parabolic curve in the same section represents the diagram of bending moments for the footing, and this diagram determines the height of the upper layer of concrete, in which a $\frac{1}{2}$-in. steel plate 24 in. square was placed to receive the four vertical bars of the column reinforcement.

36. Footings of Brick Walls.—The footings of the outside walls and the party wall are of concrete, reinforced by longitudinal and transverse bars varying from $\frac{3}{4}$ in. to 1 in. diameter, and spaced from 4 in. to 9 in. apart according to the loads to be carried. These bars are placed near the end surface of the footing, at a distance equal to about one-sixth of the thickness of the concrete.

37. Column Details.—The columns are continuous from the bottom to the top of the building, the vertical bars of the reinforcement being jointed at the distance of

6 in. above each floor level by means of loosely fitting sleeves of gas barrel, into which the ends of the bars were inserted and the annular spaces filled with cement grout. For the purpose of ensuring good bearing of one bar upon another the end surfaces of the bar were hammered square.

By the transverse and vertical sections in Fig. 40 it will be seen that the reinforcement of each column consists of four vertical bars connected at intervals by $\frac{5}{8}$-in. diameter bands cross tied with wires.

The vertical bars in the ground floor columns are of $2\frac{9}{16}$ in. diameter, the bars for the lengths above being reduced in diameter in accordance with the diminution of load, as indicated by the vertical section in Fig. 40. Throughout the columns the concrete consists of 1 part of Portland cement, 2 parts of sand, and 2 parts of washed gravel passed through a $\frac{1}{2}$-in. ring.

38. Beam Details.—Fig. 43 shows details of the

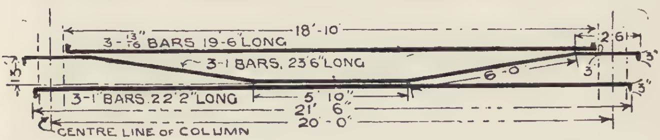


FIG. 43.—Main Beam Reinforcement.

reinforcement in one of the main beams, and it will be seen that the bars form the kind of truss usually adopted in the Hennebique system. The tension reinforcement consists of three 1 in. bars, and the compression reinforcement of three $\frac{13}{16}$ in. bars. There are also three bent bars of 1 in. diameter, to provide for continuous girder action.

A novel feature of the reinforcement is that the three bars in each truss are connected by means of electrically welded network, 4 in. by 6 in. mesh, one net enveloping each set of reinforcing bars for one-third of the span at each end. The three separate sets of reinforcement are similarly connected by means of another net extending over the centre third of the span. The nets were bent over and wired to the compression bars, thus preventing

these from buckling in an upward direction under heavy load. All the reinforcing bars were bent at the ends, for a length varying from 1 in. to 3 in., to the form of a right angle, so as to increase the bond between the concrete and the steel.

39. Column Moulds.—In building up the columns the reinforcement was first erected, and the moulds were then set up, clamped, and carefully plumbed and squared with the building.

Details of the moulds will be seen in Figs. 44 to 47. They consisted of square boxes of 1¾ in. pine boards screwed together with 2 in. by 6 in. cleats, and bound at intervals by 2 in. by 4 in. clamps bolted at the corners, the ends of the clamps being wired as shown in Fig. 44. The

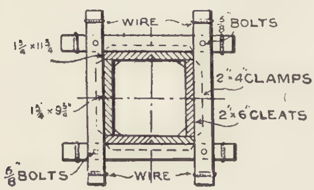


FIG. 44.—Section on AA (Fig. 46).

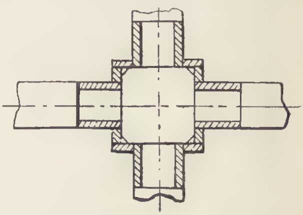


FIG. 45.—Section on BB (Fig. 46).

Column Moulds.

clamps were fixed by driving wedges between them and the cleats.

The moulds were constructed so that they could be lengthened or shortened to suit the heights of the different storeys of the building.

Fig. 45 is a cross section of a column mould at BB, and shows the arangement for the connection of longitudinal and transverse beams. At the capitals of the columns bracket reinforcement, consisting of four ⅝-in. bent bars, was placed (see Fig. 40) for connection with the lower reinforcement of the beams, and to afford more adequate support for the beams as well as to provide for shear.

Concrete, mixed very wet, was poured into the moulds, and the greatest care was taken to ensure its penetration

to all parts of the mould by stirring and tamping down the material between the bars.

40. Beam Moulds.—The beam moulds were next set up in accordance with the arrangement represented in Figs. 46 and 47.

The beam moulds were connected at each end to the column moulds, where they received additional support from 2 in. by 6 in. timber struts, and were supported further by three 4 in. by 6 in. timber posts placed on wedges at floor level, and diagonally braced with 1 in. by 6 in. boards, as indicated by dotted lines in Fig. 45.

41. Floor Slab Centring.—As soon as the beam moulds had been fixed in position the floor centring was erected. This was made in four parts for each panel, to facilitate handling. The floor surface of the moulds consisted of boards simply nailed one alongside the other. Owing to the weight of the floor, estimated at about 78 lb. per square foot, it was considered necessary to apply an adjustable centre support below each floor panel, as indicated by dotted lines in Fig. 47.

42. Fixing Beam and Slab Reinforcement.—In every case the beam reinforcement was assembled and tied together on the ground, being afterwards hoisted and placed in the moulds. To keep the lower bars from touching the bottom of the mould, the reinforcement was tied to small trestle frames provided at intervals along each span; and to preserve the predetermined lateral spacing, sets of wedges were placed between each of the three trusses, each formed by one set of the reinforcing bars. The trestles mentioned were afterwards used as supports for plank gangways required for the purpose of wheeling and dumping concrete to form the floor slabs.

After the floor centring had been prepared the reinforcing bars for the floor slab were distributed to different points, and laid by a gang of labourers upon marks previously made on the boards of the floor centring. Two lathers followed the labourers to wire the bars together. All bars in the outer rows—that is, those next to the beams—were tied at every intersection in each direction, and the next three rows were tied at every third intersection. The bars in

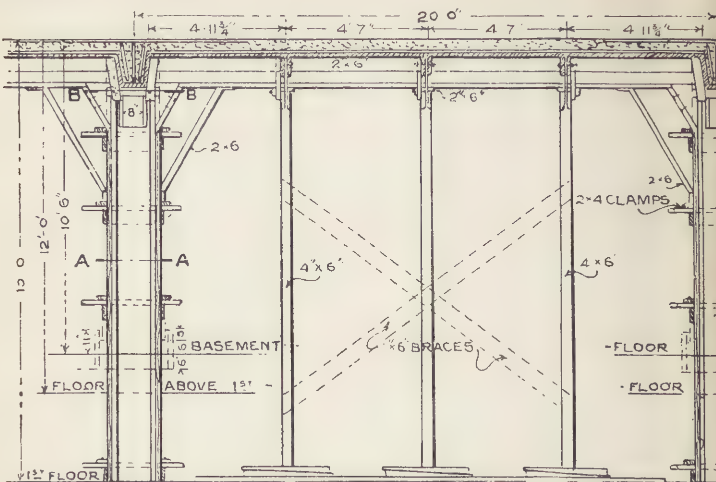


FIG. 46.—Column, Beam, and Floor Moulds (Elevation).

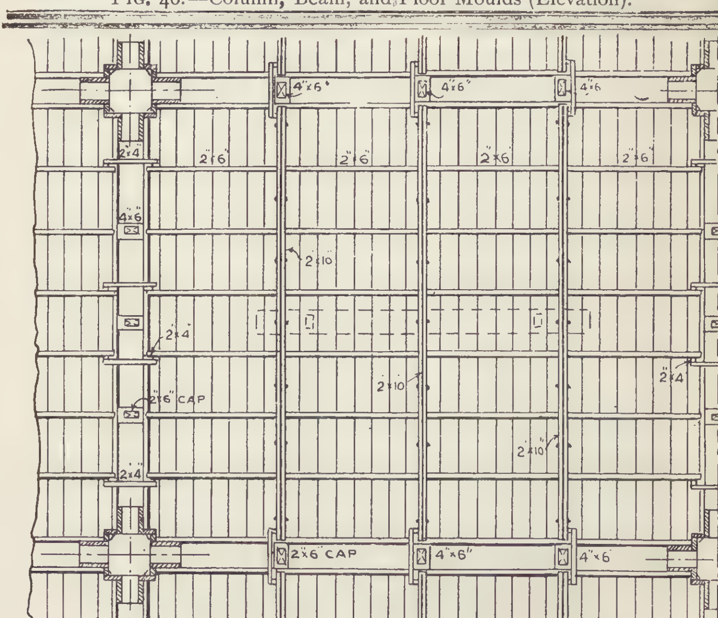


FIG. 47.—Column, Beam, and Floor Moulds (Plan).

the middle of the panel were simply laid along the marked lines. All the floor bars were wedged up about 1 in. by means of stones. Finally, a series of bars termed "negation bars" was laid across the upper reinforcement of the beams, and tied down to the floor bars on each side. These bars were designed to resist negative moments, causing tension in the upper part of the floor slab, and to prevent the upper reinforcement of the beams from buckling in an upward direction under excessive strain. All the bars of the beam and floor reinforcement which ended at the outer walls of the building were bent for about 2 in. at the end to afford satisfactory anchorage, and those for the beams were carried from one-half to two-thirds of the thickness of the brickwork into the wall.

43. **Moulding Beams and Slabs.**—Having prepared the reinforcement in the manner described, the floor beams and slabs were moulded. All the concrete for this portion of the work was in the proportions of 1 part of Portland cement, $2\frac{1}{4}$ parts of sand, and $2\frac{1}{4}$ parts of washed gravel. For the beams a very wet mixture was adopted, and the material was thoroughly stirred and shaken after pouring. For the floor slabs a somewhat drier mixture was used, so as to permit the slab to be tamped. As the concreting proceeded the stones wedged below the reinforcing bars were removed.

After tamping, the floor was levelled off with a straight edge, and then presented the appearance of a single monolithic slab broken only by necessary openings and by the projection of the column reinforcement, the bars protruding about 6 in. above the surface.

44. **Formulæ used for Floor Slabs.**—The floor slab was considered as a beam fixed at each end, the formulæ used being as follows:—

For the outer panels

$$M = wl^2 \div 12 \qquad (1)$$

For the inner panels

$$M = wl^2 \div 24 \qquad (2)$$

Where

M = Bending moment.

w = Dead and live load in pounds per square foot.

l = Span in feet.

Formula (1) gives the maximum bending moment which occurs at the ends of an *encastré* beam, and formula (2) gives the bending moment which occurs at the centre of a similar beam.

The thickness (d) of the floor slab was calculated by the rule $d = \sqrt{M \div 1000}$ . . . . (3)

The area (a) in square inches of steel reinforcement in each direction per foot width of the slab was determined by the rule $a = 0.075\ d$. . . . . (4)

In using rule (3) the bending moment, as calculated by formula (1), was taken, the floor slab being made of the maximum thickness so ascertained.

In the building under consideration all the floor slabs, except that over the third storey, are 6 in. thick, and by formula (4) the area of reinforcement per foot width in each direction is

$$a = 0.075 \times 6 = 0.45 \text{ sq. in.},$$

and for a cross section of 12 in. wide by 6 in. thick the total area of reinforcement is

$$a + a = 0.45 + 0.45 = 0.9 \text{ sq. in.}$$

45. Concrete Data.—For convenient reference the proportions and consistency of the concrete used for different classes of work in this building are collected in the subjoined table:—

Ref.	Class of Work.	Proportions of Concrete.			Consistency.
		Portland Cement.	Sand.	Aggregate.	
		Parts.	Parts.	Parts.	
Art. 35	Column Bases (bottom)	1	4	..	Medium.
	,, (top)	1	2	3 crushed Limestone gauged $1\frac{1}{2}$ in.	,,
,, 37	Columns.	1	2	2 washed Gravel gauged $\frac{1}{2}$ in.	Very wet.
,, 43	Beams.	1	$2\frac{1}{4}$	$2\frac{1}{4}$,, ,, ,,	,,
,, 43	Floor Slabs.	1	$2\frac{1}{4}$	$2\frac{1}{4}$,, ,, ,,	Medium.

CHAPTER IV

THE GREAT WESTERN RAILWAY GOODS STATION AND WAREHOUSE, BRISTOL—THE NORTH-EASTERN RAILWAY GOODS STATION AND WAREHOUSE, NEWCASTLE-ON-TYNE

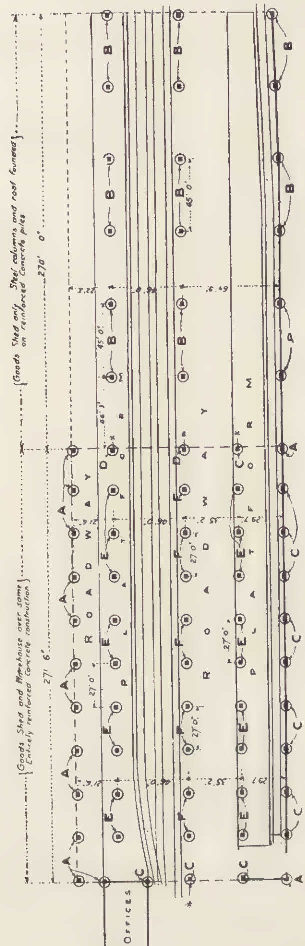
THE GREAT WESTERN RAILWAY GOODS STATION AND WAREHOUSE, BRISTOL

46. Dimensions and Accommodation.—The block of buildings here described in part includes a goods shed 541 ft. long by 133 ft. wide; a warehouse 271 ft. long by 133 ft. wide, built over a portion of the goods shed; and an office building of two storeys at one end. Fig. 48 is reproduced from a photograph taken during construction, the projecting part being the office building.

FIG. 48.—Great Western Goods Station, Bristol.

For about half its length the goods shed is practically open along one side and the two ends. It is provided with

two island platforms, and through it run four railway tracks and two roads for horse-drawn vehicles. The entire block



A. Two piles 14 in. square.
B. Two " 12 "
C. Three " 14 "
D. Four piles 14 in. square.
E. Five " 14 "
F. Six " 14 "
* Columns 18 in. square supporting floor of warehouse above the Station.

FIG. 49.—Plan of the Goods Station.

of buildings is founded on reinforced concrete piles, beams, and slabs. For about half its length the superstructure of the goods shed is of steel, but the other half, surmounted by the warehouse, was constructed together with the warehouse itself entirely of reinforced concrete on the Hennebique system, under the supervision of the railway engineers. The adoption of reinforced concrete was chiefly due to the economy and rapidity of construction rendered possible by the use of that material.

47. Foundations.—As a portion of the site occupied by the buildings was formerly a timber dock, the bearing power of the ground was an extremely doubtful quantity, and for this reason it was decided by the engineers to employ reinforced concrete foundations, supported on piles of the same material.

Fig. 49 is a plan of the entire building, wherein the positions of the piles are indicated, and Fig. 50 is a transverse section through the goods shed and warehouse which will serve to make clear the design of the foundation system.

The piles, 274 in number, vary in sectional area from 12 in. to 14 in. square, and are driven in seventy-four groups as stated below. The letters used correspond with those in Fig. 49.

A,	14	groups of	two	14-inch	square	piles.
B,	18	„	two	12	„	„
C,	13	„	three	14	„	„
D,	2	„	four	14	„	„
E,	18	„	five	14	„	„
F,	9	„	six	14	„	„

About 3 ft. below rail level the individual piles of each group are connected by reinforced concrete column bases, as illustrated in Fig. 50, the bases being connected in turn by reinforced concrete beams, so that the foundations really constitute a completely framed structure.

Ninety 12 in. square piles were required for the foundations of the steel superstructure of the goods shed, and one hundred and eighty 14 in. square piles for the reinforced concrete portion of the building.

48. Pile Driving.—The average length of the piles is 32 ft. They were driven by means of a 2-ton monkey vith

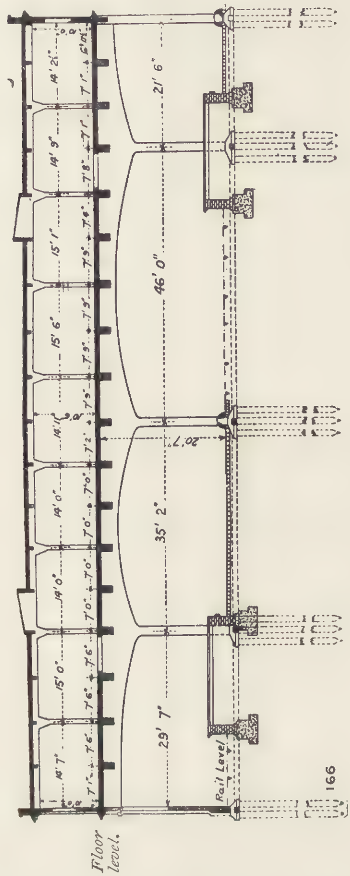


FIG. 50.—Transverse Section of the Goods Station.

a drop of 4 ft., about thirty blows per minute being given. During the operation of driving the upper end of each pile

was protected by a cast steel cap with a filling of sawdust to distribute the force of the blows, and a timber dolly was also employed as usual.

49. Main Columns.—At one side of the building a flank wall extends for the full length of 541 ft., and in the portion beneath the warehouse this wall is strengthened by piers measuring 1 ft. 9 in. by 10 in., spaced 7 ft. apart centre to centre. In addition to the wall piers there are four longitudinal rows of columns, the spacing of the columns in each row being 27 ft. from centre to centre, while the spacing between the several rows of columns and piers is 29 ft. 7 in., 35 ft. 2 in., 46 ft., and 21 ft. 6 in., as indicated in Fig. 50.

With the exception of four columns which are 18 in. square, all the main columns have the transverse dimensions of 18 in. by 24 in. The proportion of reinforcement to concrete varies according to the load, but the arrangement in every case is similar to that represented in Fig. 51, all the columns being protected by angle bars at the corners.

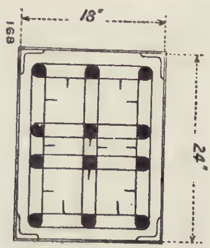


FIG. 51.

The column construction generally resembles that described in Articles 7 and 31.

50. Beams and Floor Construction.—Owing to the great weight to be carried by the warehouse floor, and the considerable spans between supports, the main arched beams running across the building are of exceptional strength. Fig. 52 is an interior view illustrating the bold and graceful lines of the arched main beams.

These beams are 1 ft. 9 in. wide by 8 ft. $5\frac{1}{2}$ in. deep at the springing, and 3 ft. $5\frac{1}{2}$ in. deep at the crown, including the thickness of the floor slab. The dimensions were made uniform for the sake of appearance, but the percentage of reinforcement is varied in accordance with the spans.

Fig. 53 is a section showing the arrangement of the steel bars and srirrups in the 35 ft. 2 in. span of a main transverse beam.

FIG. 52.—Interior of the Goods Station.

Fig. 54 is a photographic view showing the moulds used for the construction of the beams.

The secondary beams, shown in section by Figs. 50 and 53, were built monolithic with the main beams, being spaced at intervals ranging from 6 ft. 11½ in. to 7 ft. 9 in. centre to centre. Their dimensions are 12 in. wide by 2 ft. 2½ in. deep, including the floor slab, which is 5½ in. thick, and reinforced in the usual manner by longitudinal and transverse bars.

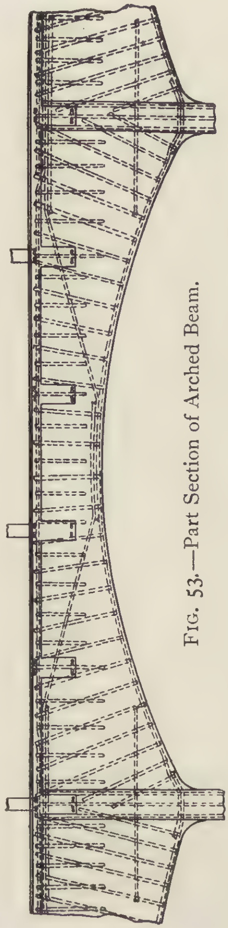


FIG. 53.—Part Section of Arched Beam.

Fig. 55 illustrates the manner in which the main and secondary beams are incorporated, and shows the stirrups of these members projecting above the centring for incorporation in the concrete of the floor slab.

51. Wall and Roof Construction.—The walls of the warehouse are 7 in. thick, and the flat roof is supported by columns with the cross section of 8 in. square rising from the main beams. The columns are spaced 27 ft. apart longitudinally, and from 14 ft. to 15 ft. 6 in. apart transversely from centre to centre.

Main and secondary beams of small dimensions connect the heads of the columns and carry the roof slab, which is 3½ in. thick, and is designed for a load of about 62 lb. per square foot.

52. Concrete.—All concrete used in the building was made with granite chippings and Pennant stone, the proportions being 1 part of Portland cement to 4 parts of aggregate for columns and walls and 1 part of

Portland cement to 5 parts of aggregate for beams, floor slabs, and the roof slab.

FIG. 54.—View of Column and Beam Moulds.

53. Floor Load and Tests.—The warehouse floor is

proportioned for a superload of 4 cwt. per square foot, and according to the engineer's specification it was required to

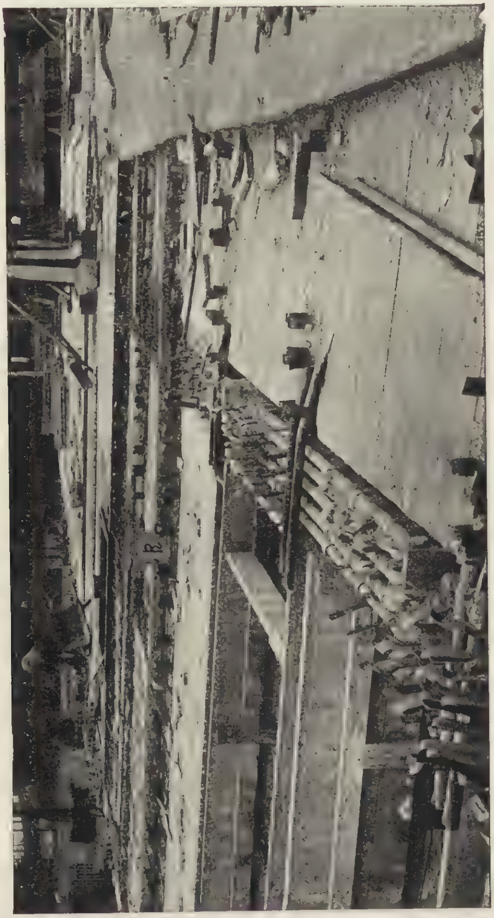


Fig. 55.—View of First Floor during Construction.

be capable of carrying a test load of 6 cwt. per square foot

without permanent deflection, the elastic deflection permitted being equal to $\frac{1}{600}$ of the span.

On completion of the building two load tests were made at first floor level.

Test No. 1, which extended over 11 days, was conducted upon a main beam of 46 ft. span, the load being placed upon a surface of 418.5 square feet.

The maximum load of 112 tons 11 cwt. (equal to 6 cwt. per square foot was applied on the eighth day, and the maximum deflection was then only 0.244 in. as compared with the permissible deflection of 0.92 in.

Test No. 2, which extended over 3 days, was conducted upon a secondary beam of 27 ft. span, the load being placed upon a surface of 202.5 square feet.

The maximum load of 60 tons 15 cwt. (equal to 6 cwt. per square foot) was applied at the end of the first day and the maximum deflection was then only 0.086 in., as compared with the permissible deflection of 0.54.

THE NORTH-EASTERN RAILWAY GOODS STATION AND WAREHOUSE, NEWCASTLE-ON-TYNE

54. General Description.—This building, completed in August 1906, constitutes the finest and most impressive demonstration hitherto available of the advantages of reinforced concrete construction to railway companies. In point of size the station is smaller than the warehouses described in Chapter I. It possesses other characteristics, however, that are far more striking, the most noteworthy being the exceedingly heavy loads carried by the columns and floor systems. How great these are may be realised by the statement that, in addition to the dead load, the main floor is designed to carry the moving load of six goods trains, and to withstand the vibratory stresses due to the working of heavy cranes, turntables, and other machinery.

The new building, of which Fig. 56 is a photographic view taken from the south-west, is situated at New Bridge Street, near the former terminus of the Blyth and Tyne Railway Company, now merged in the North-Eastern Railway system.

It forms one item of a comprehensive scheme for looping up the various connections of the North-Eastern Railway in and about Newcastle. New Bridge Street passenger station is no longer to be a terminus, as the coast lines have been joined up with the main lines north and south running into the Central station. To provide for the execution of the project, it was necessary to acquire and demolish a large area of property in the Manors district, part of the land so rendered available being reserved for the new lines, and the remainder for the purposes of a new goods depôt, in which the building here under consideration is a most important part. Fig. 57 is a view showing the north end of the goods station.

In addition to station accommodation, provision is made for warehousing general merchandise, and for the storage of flour on a large scale. The main dimensions of the building are 430 ft. long by 178 ft. 4 in. wide by 83 ft. 4 in. high from the basement floor to the top of the parapet.

55. Equipment of Low and High Level Stations.—Fig. 58 is a plan of the basement floor, which is designed for use as a low-level goods station with four tracks for trains, and ample space for dealing with inward and outward goods. Access is given to the station by two waggon hoists (HH) and a subway leading from the goods yard. The station is also furnished with a waggon traverser and four turntables.

Fig. 59 is a plan of the ground floor, designed as a goods station in direct communication with the main line of the North-Eastern Railway. This floor has six railway tracks and three platforms, as well as eight turntables and numerous capstans for the manipulation of waggons.

For dealing with merchandise there are provided in the basement two electric cranes DD revolving around pillars, and two overhead revolving and travelling electric cranes EE. On the ground floor there are two radial electric wall cranes FF, and two overhead revolving and travelling electric cranes XX.

At the south end of the station there is a spiral staircase, constructed entirely of reinforced concrete, leading from the ground floor to the upper storeys. Fig. 60 is a photo-

graphic view of this detail, which consists of a cylindrical

FIG. 56.—New Bridge Street Goods Station, Newcastle-on-Tyne. View from the South-West.

column, about $7\frac{7}{8}$ in. diameter, from which the treads project without external support of any kind.

56. Equipment of Warehouse.—The upper floors are intended for storage purposes, and a portion of each upper

FIG. 57.—Exterior of Goods Station, North End.

floor is prepared with inclined floors for the reception of flour. The flat roof is available for crates and packages

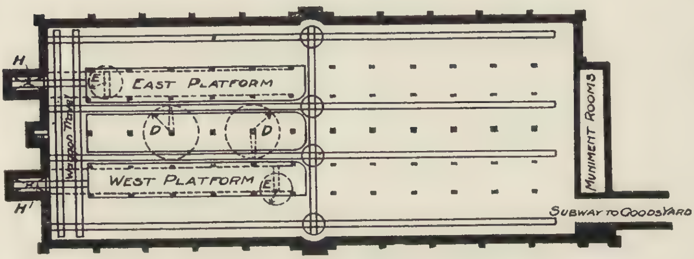


FIG. 58.—Plan of Low Level Station.

not requiring protection from the weather. Figs. 61 and 62 are plans of the first and second floors, and Figs. 63 and 64

are sections through the automatic floor store, and hoists, respectively.

On each of the upper floors fourteen doorways are provided (see Figs. 61 and 62), over which electric hoists are fitted so that goods can be unloaded from or loaded into railway trucks brought alongside the building. These hoists are protected from the weather by cantilever hoods projecting 15 ft. The hoists MM traverse from N to O (see Fig. 63), and are designed for the delivery of flour and grain into the automatic store, while the hoists PP raise and lower only. There are also four conveyor hoists QQQQ travelling on tracks fixed immediately below the

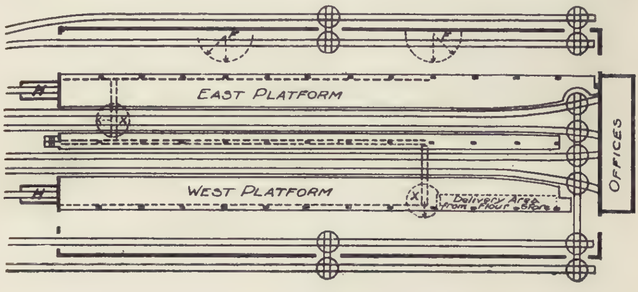


FIG. 59.—Plan of High Level Station.

roof. Outside the building these hoists are housed in cantilever towers projecting about 15 ft. from the face of the outer walls. To increase the scope of the hoists the floors of all the upper storeys are slotted so as to permit the chain and hook to be passed down to platform level in the ground floor station. Consequently, packages can be slung on any floor, traversed to the light wells and other openings, and transferred to any other floor, or deposited in trucks on the railway lines in the yard outside.

57. **Main Structural Features.**—While the entire building is of monolithic character, it may be regarded in a way as representing what is termed "cage" construction, for the entire weight of the walls, floors, and roof, and all superloads, are carried by a framework of girders and

columns, and transmitted by the latter direct to foundations resting on boulder clay, the safe load on which was taken at 5 tons per square foot. Figs. 63 and 64 make clear the general character of the construction.

Up to the first floor level the main framework comprises 70 wall and interior columns in five rows spaced 33 ft. apart longitudinally, while between the rows there are two spaces of 37 ft. 2 in. and two of 52 ft. each, centre to centre.

The columns are connected at each floor by wall beams or lintels, and by main and secondary girders inside the station. There are also 30 auxiliary columns in the basement and ground floor storeys for carrying exceptionally heavy local loads.

Above first floor level the framework consists essentially of 440 wall and interior columns in thirteen rows, each of which, where unbroken by light wells and the automatic flour store, contains 40 columns, spaced 11 ft. apart longitudinally and 15 ft apart transversely. The arrangement of the columns on the first floor will be seen in Fig. 61. These columns are connected at each storey by wall lintels and interior beams.

FIG. 60.—Spiral Stair.

The upper storeys represent quite a different class of design from that in the lower portion of the station, whose main columns and girders may be said to act as foundations for the upper part. Nevertheless, the two systems of construction are intimately connected to form one complete frame of enormous strength, whose rigidity is further increased by the monolithic incorporation of the floor slabs,

platforms, flat roof, and walls. The latter between the columns and lintels are merely curtain walls, and, having no weight to carry, are only 4 in. thick.

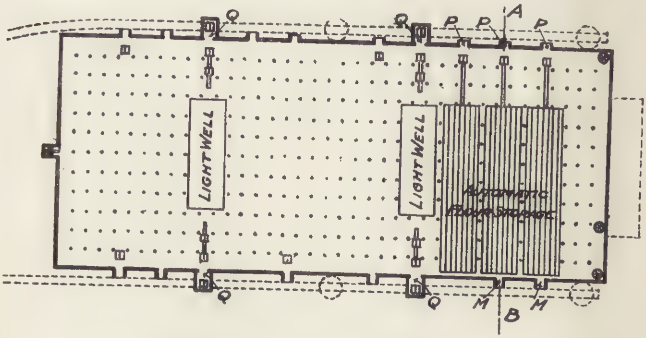


FIG. 61.—Plan of First Floor.

58. **Main Columns.**—The wall columns rest upon massive foundations of concrete, which also serve the purpose of retaining walls, and their inner faces form the

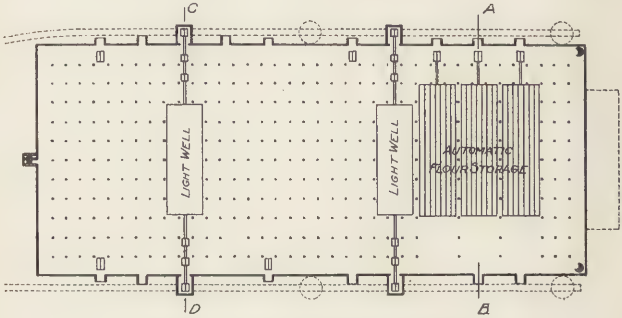


FIG. 62.—Plan of Second Floor.

side and end walls of the low-level station. The interior columns in each transverse row terminate in bases of triangular elevation. In the centre row the bases are 15 ft.

6 in. square, the two small bases on each side of the centre are 7 ft. square, and the two outer bases are 14 ft. square.

Two of the five rows of columns in the basement are

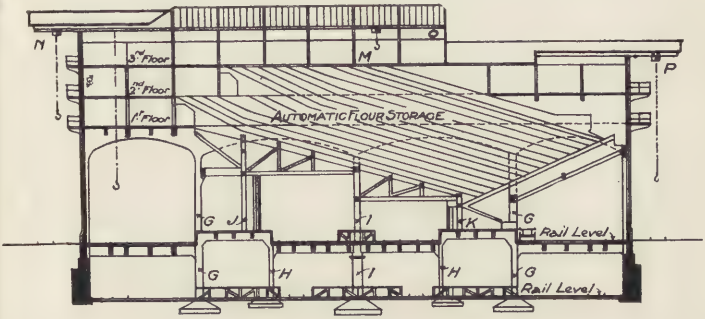


FIG. 63.—Section through Automatic Flour Store (line AB, Figs. 60 and 61).

simply for the purpose of supporting the floor of the upper station, the other three rows being carried up to the under side of the first floor.

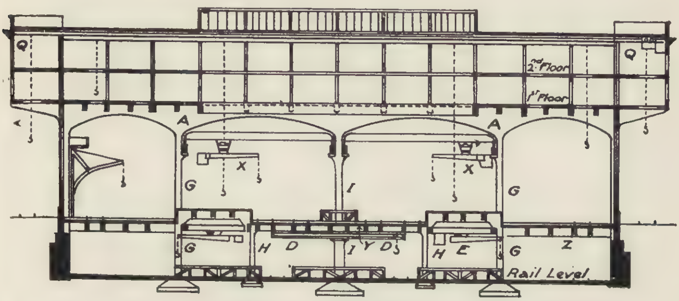


FIG. 64.—Section through Hoists (line CD Fig. 60).

The loads on the columns are widely different. For instance, on columns type H the load is only 224 tons each, on columns type G the load is 927 tons each, and

each column I in the central row has to carry the enormous load of 1,105 tons. One of these columns is to be seen at the left hand of Fig. 66.

Fig. 65 contains a sectional elevation of a central column type I, and detail drawings illustrating the arrangement of the reinforcement. Being self-explanatory, these illustrations require no comment, but it may be mentioned that

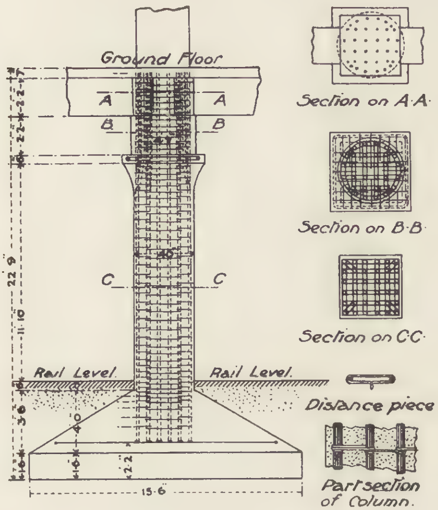


FIG. 65 —Details of Main Column.

the footing shown plain in the elevation is really reinforced by horizontal bars laid in rows at right angles to each other, and connected by vertical loops of flat steel for resisting shearing stresses. The extended capital near the top is for the purpose of carrying the rails for the travelling cranes in the lower goods station. A striking testimony to the strength of reinforced concrete is to be found in the fact that the type of column here illustrated only required the sectional area of 1,600 square inches, which is very

little more than that of a steel column of equal strength with an adequate casing of fire-resisting material.

In the upper station, columns G and I have the transverse dimensions of 24 in. by 36 in. and 26 in. by 36 in. respectively. All the columns of these types are provided with extended capitals of suitable form for carrying the rails for the travelling cranes.

59. Cantilever Construction.—Outside the eastern boundary of the site the level of the ground is considerably higher than the surface of the goods yard, and owing to the unstable character of the earth for a distance of some 200 ft. along the eastern side of the station it was considered desirable to supplement the plain concrete retaining walls by the addition of ferro-concrete construction, consisting of stanchions built up from the basement floor to the under side of the ground floor girders, these stanchions having extended bases of the same material, and between them a thin but strongly reinforced wall of ferro-concrete. The upper ends of the stanchions are tied by a beam of ferro-concrete, affording support for the main girders at ground floor level; and to prevent any tendency of the stanchions to move inward under the outer earth pressure, horizontal struts were formed in the thickness of the basement floor, reacting against the column bases in the first interior row. In virtue of this arrangement the whole weight of the building was brought to the aid of the retaining wall and stanchions in resisting the inward earth pressure. With the further object of reducing the load on the retaining walls, the main wall columns are carried by cantilever projections of the transverse girders in the ground floor. These cantilevers extend for a length of 3 ft. beyond the ferro-concrete stanchions by which they are supported.

The cantilever housings for the conveyor hoists, and the cantilever shelters over the various loading doors, are examples of remarkably bold design, and would have been thought distinctly hazardous a few years ago when engineers in this country were unfamiliar with the valuable properties of reinforced concrete.

60. Beams of Ground Floor.—The ground floor of the building is carried entirely by reinforced concrete

columns, and includes six spans, the measurements of which are 35 ft. 3 in., 24 ft. 3 in., 27 ft. 9 in., 27 ft. 9 in., 24 ft. 3 in., and 35 ft. 3 in.

The transverse beams, spaced 33 ft. apart, centre to centre, are 1 ft. 6 in. wide by 2 ft. 9 in. deep, two portions of the beams on the inner side of the outermost row of columns being raised so as to form platforms for handling merchandise. The longitudinal beams of the same floor system are 1 ft. wide by 2 ft. 3 in. deep, and the spacing varies according to the requirements of the spans, from 4 ft. to nearly 7 ft. (see Fig. 66).

All these beams are monolithic with the columns, and the whole system is connected by a floor slab 9 in. thick, forming a compression flange common to the network of beams. Upon and partly embedded in the slab are sleepers carrying the various rail tracks, and the floor surface is formed by a layer of concrete 6 in. thick.

Owing to the necessity for adopting long spans, to avoid interference with the conduct of traffic in the sub-station, the loads coming upon the beams are exceptionally heavy. This will be realised when it is stated that the ground floor was calculated for a dead load of 3 cwt. per square foot in addition to the moving load of railway traffic. As each loaded waggon may represent a dead weight of more than 42 tons, the total weight on the floor will be very large, while the dynamic effect of moving trains and the handling of heavy merchandise, and the operation of machinery will add very considerably to the strains to be resisted. To illustrate the great strength of the floor, we give in Fig. 67 a view taken beneath one of the railway turntables.

61. Beams of First Floor.—One of the most interesting features in the building is the series of arched beams spanning the distance of nearly 180 ft. from wall to wall of the high-level station, with the intermediate support of three columns. Similar beams extend across the building at intervals of 33 ft. centre to centre (see Fig. 68). The two outer spans of each series measure 37 ft. 2 in. between the centre lines of the supports, and the two inner spans 52 ft. from centre to centre. The beams measure 1 ft. 6 in. wide, 9 ft. deep at the springings, and 3 ft deep at the crown.

Fig. 66.--View in Low Level Station, showing columns carrying 1,105 tons each.

Fig. [illegible]. View of Floor beneath Turntables in High Level Station, Newcastle.

With one exception these are by far the longest span concrete-steel beams ever built in this country, and they have only been surpassed in a few isolated instances in other parts of the world. The test load specified for each beam was 400 tons.

Fig. 69 is the reproduction of a drawing that will well

FIG. 68.—Interior of High Level Station.

repay examination, and as full particulars of the construction are there given detailed description is unnecessary.

Attention may be directed, however, to the reinforcement in the arched beams, which is so disposed that no thrust is exerted against the outer walls. Three details for beam A are given on line TT. These refer to different members of the same general type crossing the building at other points. At the upper part of the drawing will be found particulars of the secondary beams of the first floor and a section showing the reinforcement of the connecting floor slab.

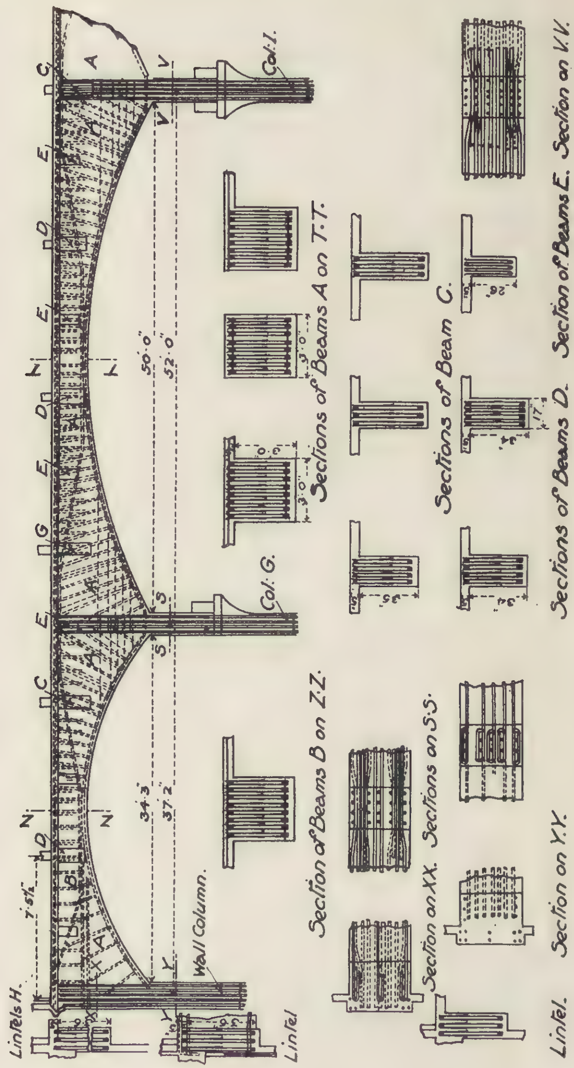


FIG. 69.—Details of Girders over High Level Station.

Another interesting series of sections at the left-hand side shows the construction of the wall beams, or lintels, carrying the weight of the curtain walls and one edge of the outermost panel of the floor slab.

The upper storeys of the new building, as shown by the transverse sections (Figs. 63 and 64), possess no features particularly worthy of remark, being carried on small columns supported by the main girders above the high-level station.

62. Upper Floor Construction.—It will be seen on reference to the plans and sections that the floor spans in the upper storeys are reduced to 14 ft. 11 in. centre to centre, by the small columns which are carried on the main longitudinal girders of the first floor. These girders are supported in turn by the arched transverse beams, and vary in dimensions according to position, some of them projecting 2 ft. 9 in. and others 3 ft. 3 in. below the under surface of the floor slab, the width varying between 15 in. and 18 in. The intermediate longitudinal girders shown in the section are 12 in. wide, and project 2 ft. 3 in. below the floor slab.

63. Automatic Flour Store.—A department occupying a space of about 500,000 cubic feet is that termed the "Automatic Flour Store." This consists, as illustrated in Fig. 63, of a series of parallel sloping floors, divided so as to constitute shoots, into which bags of flour or grain can be inserted at the second and third floor levels until the whole of the store is filled, the total capacity being nearly 17,000 twenty-stone sacks. The delivery of flour or grain is controlled by a series of levers, operated by electricity from a central station, and arranged so that the exact quantity required can be discharged upon the delivery area of the platform on the ground floor (see Fig. 59), the exact number of bags delivered being recorded by automatic registering apparatus.

Reference to Fig. 63 will show that the supports for the automatic storage consist of triangular frames and horizontal beams of different depths and spans, taking bearing on the three main columns GG and I, and upon two additional columns J, K rising from the east and west station platforms.

Figs. 70 and 71 are views showing the framework supporting the flour store and the conveyor galleries of the same department.

At their lower ends the inclined shoots discharge into electrically driven conveyors moving in an upward direction, so that the hinged door at the bottom of each shoot may offer no obstruction to the discharge of the sacks, which are transferred by the conveyor down a slope parallel to

FIG. 70.—Conveyor Galleries of Automatic Flour Store.

that first mentioned, and finally down a third slope to the delivery area, 95 ft. wide, marked on the west platform in Fig. 59.

The latticed frames or ramps supporting the automatic flour store, are of different types, one extending from the west column G to the centre column I, with intermediate support from column J, and the other continuing the same slope from column I to the auxiliary column K. The load on each of the triangular frames varies from 80 to 100 tons.

Similar frames are spaced at intervals of about 9 ft. centre to centre, under the whole length of the automatic store, which Fig. 61 shows to be divided into three sections by the main arched girders passing from wall to wall of the building. With the exception of these members the store has necessitated the suppression of all the main and secondary beams in a large portion of the building. Consequently, on the third floor there is a gap of 60 ft. by 95 ft., on the second floor one of 100 ft. by 95 ft., and on the first

FIG. 71.—Supports of Automatic Flour Store.

floor a gap of 140 ft. by 95 ft., the latter divided by two of the main arched girders.

It will be readily understood that the break of continuity in these floor systems presented serious engineering problems for solution, especially as the stipulation was made by the railway company that the construction should be such that in case of need the whole of the automatic storage accommodation might be removed, and the flooring system made continuous in each storey of the building.

Fig. 72 contains a sectional elevation of the triangular

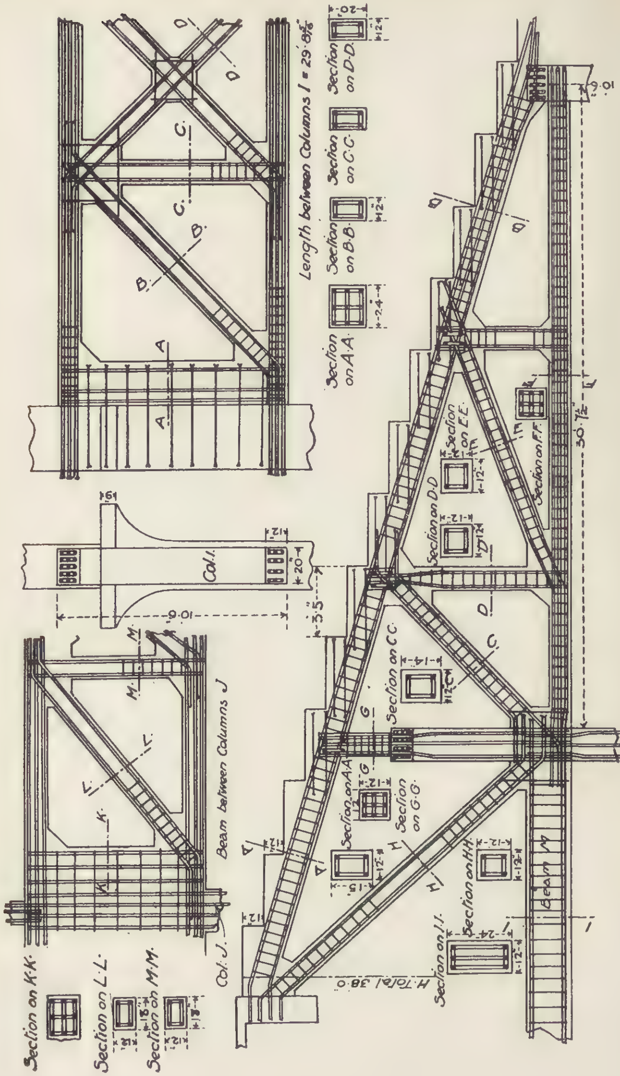


FIG. 72.—Details of Lattice Girders and Framework supporting Automatic Flour Store.

frame between the east and central main columns G and I. The total length of this frame between the columns is 50 ft., with one intermediate support at column J, about 15 ft. from the western end. At the upper end the depth of the frame is 15 ft. 6 in., and, as seen by the drawing, it tapers down almost to a point at the lower end. Several sections of the compression and tension booms, the ties and struts, are also illustrated. Attention may be particularly directed to the general disposition of the reinforcement, and to the satisfactory manner in which the bars of the different members are anchored into the concrete so as to secure the action of the whole as a genuine framed structure.

The two series of triangular frames supporting the automatic storage are connected by lattice girders of ferro-concrete, parallel with the longitudinal axis of the station. One of these girders, 1 ft. 6 in. wide by 8 ft. 6 in. deep, is to be seen in section above column J in Fig. 72. This latticed girder carries a load of 161 tons.

A still larger girder of the same type is illustrated at the right-hand top corner of Fig. 72, and in Fig. 73. This member, designed for a load of 309 tons, passes longitudinally between the central main columns I, and serves to support the ends of the upper and lower triangular latticed frames. The girder has a width of 1 ft. 8 in., a depth of 10 ft. 6 in., and a total length of 33 ft. from centre to centre of the two supporting columns. It was at first intended by the architect that this and the other girders perpendicular to the horizontal axis of the triangular frames should be solid rectangular beams of ferro-concrete, but on the recommendation of the structural engineer they were built as lattice girders, with a very considerable saving of material, and the further beneficial effect of offering far less obstruction to the diffusion of light in the high-level station.

64. Materials.—In the building of which a few of the chief features have been here described over 2,600 tons of mild steel bars were used.

The quantity of Portland cement applied in the form of concrete and otherwise amounted to 3,000 tons, the general proportions of the concrete being 1 part of Portland cement,

2 parts of sand, and 4 parts of washed gravel, both sand and gravel being obtained from the Tyne.

It should be mentioned, however, that the proportions of the concrete were varied in different parts of the work, according to the character and duty of the members, and the proportions were also varied, from time to time, with the ascertained percentage of voids in the aggregate.

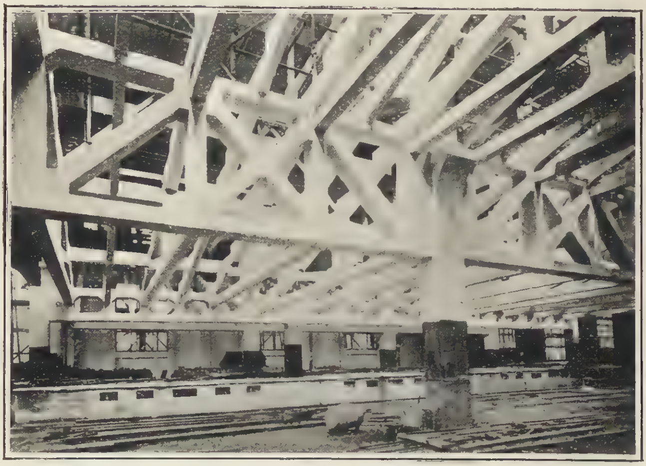


FIG. 73.—Lattice Girder Supporting Automatic Flour Store.

The surface of the concrete is rendered in cement mortar, giving a finish closely resembling that of fine stone.

The surface of the flat roof is covered with "Ruberoid," a material whose basis is similar to indiarubber, having the same elastic and waterproof properties, while it is free from the disadvantage of oxidation, which causes real rubber to become hard in course of time, and so to lose its efficiency. Between 7,000 and 8,000 sq. yds. of "Ruberoid" were used on the roof of the goods station.

65. Tests.—In March 1905 two panels of the floor were tested under the supervision of the designer. The

first was one of the panels over the beam Y, and the second one of those over the beam Z (see Fig. 56).

On the basis of 336 lb. per square foot, the test load for the 27 ft. 9 in. span was equal to 460,000 lb.; but as a matter of fact, the specified weight was exceeded by 45,000 lb., the total weight applied being 505,000 lb., or nearly 370 lb. per square foot. In spite of this the maximum deflection at the centre of the supporting beam was only 0.835 in.

In the case of the 35 ft. 2 in. span, the specified test load was about 573,000 lb., an amount that was exceeded in the official trial by 37,000 lb., the total load being 610,000 lb., or nearly 360 lb. per square foot. Nevertheless the maximum deflection of the beam at the centre was only 0.312 in.

The measuring instruments employed for the purpose of registering the deflection during the loading and unloading of the floor panels indicated that the beams began to return to their original form as soon as unloading was commenced, thus demonstrating the positive elasticity of the construction.

The building was designed by Mr. William Bell, F.R.I.B.A., the architect to the North-Eastern Railway Company, in accordance with the Hennebique system, the mechanical equipment having been designed by and installed under the direction of Mr. Charles A. Harrison, M.Inst.C.E., the engineer to the company. All details of the ferro-concrete construction were prepared by Mr. L. G. Mouchel, M.Soc.C.E. (France), whose resident engineer was Mr. T. J. Gueritte, of Newcastle-on-Tyne, and the building contractors were Messrs. Joseph Howe & Co., of West Hartlepool. All the steel used on the works was supplied by the Consett Iron Co., and the Portland cement by Messrs. I. C. Johnson & Co., of Gateshead.

CHAPTER V

A One-Storey Factory Building near New York—Printing Works in London — A Five-Storey Factory Building in Philadelphia — Business Premises in Southampton

A ONE-STOREY FACTORY BUILDING NEAR NEW YORK

66. General Description. — The building here described is typical of the Wight-Easton-Townsend system of reinforced concrete. It is 312 ft. long, with a minimum width of 52 ft. and a maximum width of 112 ft. Beneath one of the workshops, measuring 142 ft. long by 65 ft. wide, a basement has been constructed, a portion of which is shown in section by Fig. 74. The roof of the entire building consists of a concrete-steel slab with girders of the same material, and over a part of it are situated three tanks for the storage of water.

The building was erected on a site in Long Island City, New York, where the ground is of alluvial character, and in fact little better than mud. Consequently, pile foundations were absolutely necessary, the piles being driven in groups, upon each of which a mass concrete footing was formed, and upon the footing a concrete pier, as represented in Fig. 74.

67. Columns and Walls.—The footings were spaced 16 ft. apart longitudinally on lines corresponding with those of the outer walls, and in one portion of the building additional footings were made to afford a longitudinal row of supports for interior columns. The lateral spacing of the footings varies from 33 ft. to 52 ft. in different parts of the building.

The columns, rising from the tops of the concrete piers,

support the main floor and the roof. The exterior columns are incorporated with the walls, forming pilasters either on

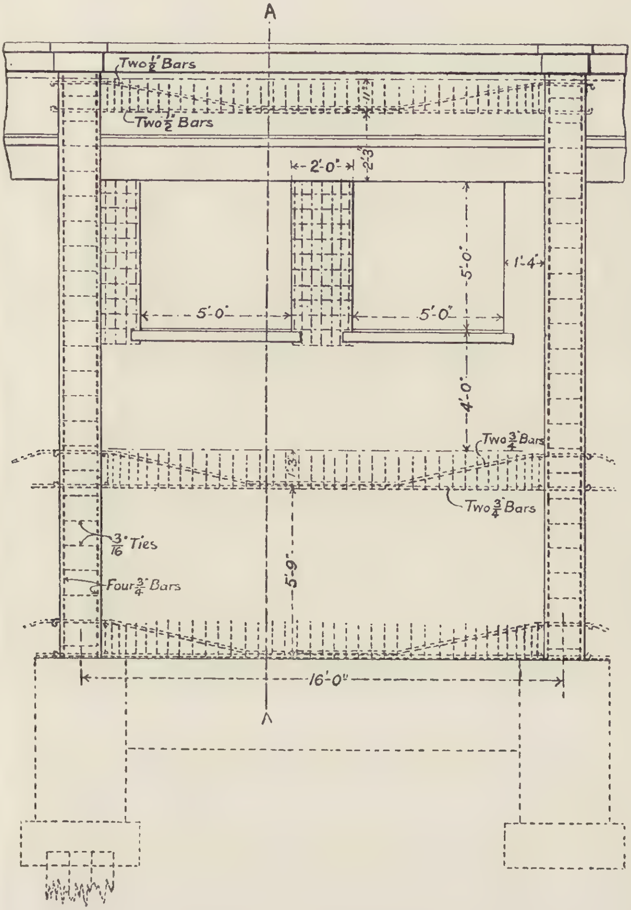


FIG. 74.—Part Section of Factory Building near New York.

the outside as in Fig. 75, or on the inside as in Fig. 76, while the interior columns serve as intermediate supports for the girders.

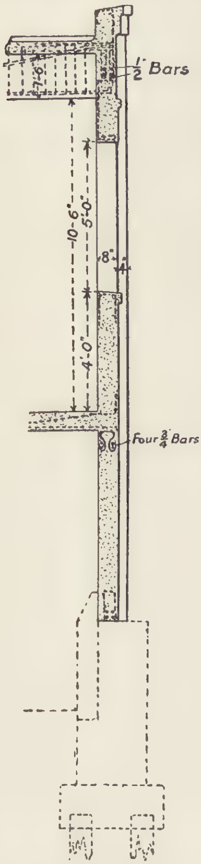


FIG. 75.—Section AA (see Fig. 74).

The exterior columns have a section of 16 in. by 12 in., and are reinforced by four vertical bars, with $\frac{3}{16}$-in. diameter horizontal ties at intervals of 12 in. measured vertically. As shown in Fig. 74, the vertical reinforcement is composed of $\frac{3}{4}$-in. diameter bars, but in some of the columns it was necessary to employ $1\frac{1}{2}$-in. diameter bars as vertical reinforcement. All the concrete used in column construction was mixed in the proportions of 1 part Portland cement, 2 parts sand, and 4 parts broken stone.

A section of the exterior wall at the basement will be found in Fig. 75, from which it may be seen that, commencing at the top level of the concrete piers, the design is that of a curtain wall. To ensure adequate support between the piers, the wall is provided with reinforcement consisting of four steel bars, two straight and two bent, and of thirty-nine vertical stirrups for resisting shear. This reinforcement really converts the lower part of the wall into a strong beam. At the level of the main floor the floor slabs are carried by girders formed in the thickness of the wall, the reinforcement being similar to that employed in the case of the walls at basement level.

Apart from the columns and girders incorporated in them, the walls are reinforced throughout with sheets of steel netting placed near the inner surface, the edges of the sheets being bent back to U-shaped section at the various

window openings. This netting has strands of No. 9 gauge wire with 6-in. by 4-in. meshes, secured at the intersections by pieces of No. 9 gauge wire. Sheets of the same material, also bent to U-shape, are embedded below the window sills, and immediately under the roof slab the wall, for a depth of 11 in., is converted into a girder by the addition of longitudinal and vertical reinforcement, generally similar to that at the lower levels. The details in question are clearly shown in Figs. 74 and 75.

In Fig. 78 may be seen particulars of a wall bracket suitable for the attachment of a pedestal for shafting. The formation of brackets in this way saves hacking the walls about, and affords a far better connection than would be given by the usual bolts and nuts. Although this bracket may seem an insignificant detail, it is sufficient to suggest the great adaptability of concrete-steel to architectural and other requirements.

The walls are 6 in. thick, and the concrete was mixed in the proportions of 1 part Portland cement, 2 parts sand, and 5 parts cinders.

68. Floor Construction.—In the basement the floor consists of a bed of simple concrete 6 in. thick, deposited on rammed and levelled earth.

That portion of the floor which is above the basement consists of a continuous concrete-steel slab 7 in. thick, the reinforcement being applied in the form of two sheets of steel netting, one sheet perfectly horizontal and the other bent up towards the lines of support. The edges of the sheets are folded at right angles to afford satisfactory anchorage. A part of the floor slab is shown in the section AA, Fig. 75. The weight of the main floor is carried entirely by the columns and girders, the latter term including the girders formed in the substance of the outer walls.

69. Roof Construction.—One of the most interesting features is to be found in the roof construction. The upper surface of the slab has a slope of about 1 in 48, and the slab is divided into panels, approximately 16 ft. square, by the transverse and longitudinal girders. Over some of the workshops the slab is 4 in. thick, but generally

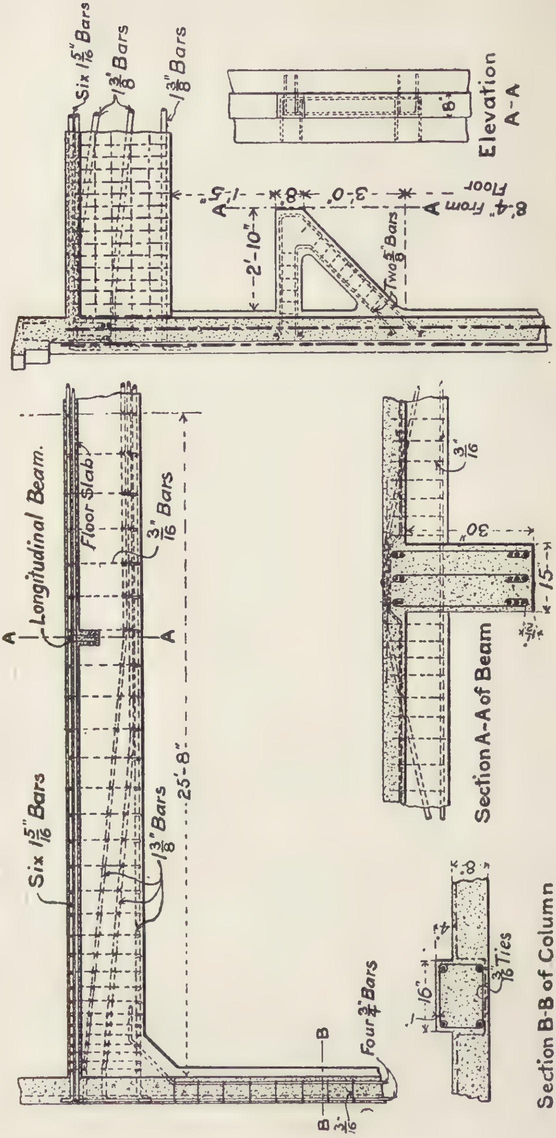


Fig. 76.

Fig. 77.

Fig. 78.

Details of Walls and Roof Girders.

the thickness is 6 in. Reinforcement is provided by two sheets of steel netting, as in the case of the main floor slab.

The transverse girders vary in dimensions and span at different parts of the building. Over one room their dimensions are 10 in. wide by 24 in. deep, over the boiler-room they are 12 in. wide by 24 in. deep (see Fig. 79), while elsewhere they are 15 in. wide by 30 in. deep. The last-mentioned dimensions apply to the girders illustrated in Figs. 76 to 78, these having the exceptionally long span of 52 ft. centre to centre, or 50 ft. 2 in. clear span between supports. These members are reinforced by six horizontal $1\frac{5}{16}$-in. diameter bars in the compression area, three horizontal $1\frac{3}{8}$-in. diameter bars in the tension area, $1\frac{1}{2}$ in. above the bottom surface of the concrete, and two sets of three bent bars of $1\frac{3}{8}$-in. diameter. The upper and lower horizontal bars are connected by vertical bars $\frac{3}{16}$-in. diameter, the ends being turned over the horizontals. The vertical bars are wired to the bent longitudinal bars, and the whole reinforcement formed a rigid framework before it was placed into the mould. The vertical bars for resisting shearing stresses are spaced apart at distances increasing from $4\frac{1}{2}$ in. at the supports to 18 in. at the centre of the beam. The section AA in Fig. 77 gives details of the reinforcement in the longitudinal beam shown in the upper drawing. The longitudinal girders measure 6 in. wide by 12 in. deep in some cases, and 8 in. wide by 12 in. deep in others.

Three panels over the boiler-room are covered by concrete-steel water-tanks, particulars of which are shown in Fig. 79. It will be observed that the tank at the left side of the figure is about 14 ft. wide by 12 ft. deep, inside measurement. The walls of the tank are formed by vertical slabs of reinforced concrete, with stanchions at intervals, generally similar in design to the joists used in the floors.

Owing to the weight of the tanks and of the water contained therein, the stress in the compressive areas of the girders was found by calculation to be greater than the compressive resistance of the concrete. Hence it became necessary to add horizontal reinforcing bars along the upper

flanges. Similarly, owing to the high stress in the tension areas, it was requisite to increase the amount of reinforce-

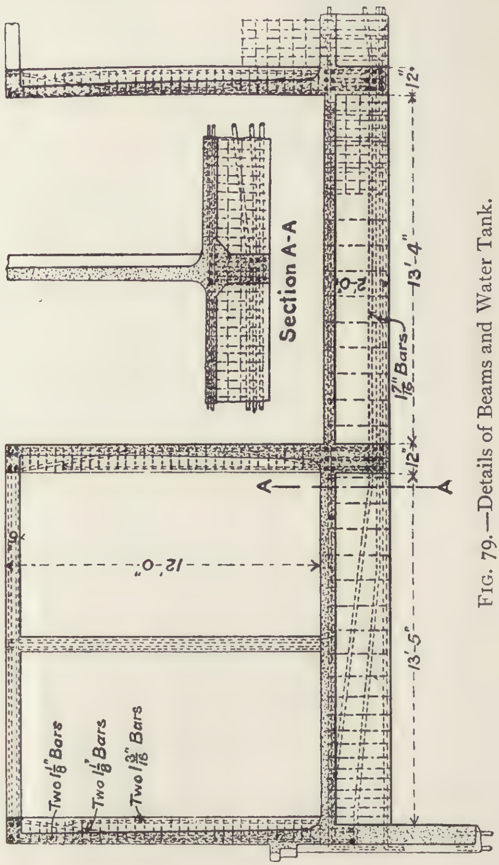


Fig. 79.—Details of Beams and Water Tank.

ment. By reference to the two sections in Fig. 79 it will be observed that nine $1\frac{7}{16}$-in. bars are used in the tension areas of each type of beam, and that eight bars of equal

diameter are employed in the compression area of one beam and six in the corresponding area of the other.

70. Strength of Materials. — From the foregoing particulars a good general idea may be obtained of the main features of the construction exemplified in this building. We may point out, however, that the design of the walls and floor slabs is of distinctive character, and that exceptional provision has been made for withstanding shearing stresses in all the members subject to flexure, as well as for obviating voids in the concrete and other defects due to carelessness or want of skill on the part of the workmen.

The following were the chief data taken into account by the designers :—

Ultimate compressive strength—	
Cinder concrete	1,500 lb. per sq. in.
Stone concrete	2,166 „
Factor of safety	6
Steel	80,000 lb. per sq. in.
Factor of safety	4
Ultimate tensile strength—	
Concrete	0
Steel	80,000 lb. per sq. in.
Facter of safety	4

Adhesion between concrete and steel = 80,000 lb. for a bar having a length 26 times its diameter.

With the object of verifying the correctness of the calculations three test beams were made by the engineers, these beams having a clear span of 20 ft. The general result of the trials was that each beam showed cracks at the lower surface of the concrete when the load was approximately equal to the calculated breaking load.

71. Moulds and Method of Construction.—The moulds employed during construction were of very simple design, as may be seen by inspection of Figs. 80, 81, and 82. Part of an interior column mould is illustrated in Fig.

80. The boards forming the three fixed sides were carefully planed and secured by vertical battens, and the boards finally constituting the fourth side were nailed on, one after the other as the concrete was deposited. The first stage in the making of a column was to put the vertical reinforcement in place, with the temporary horizontal ties at the top as shown in the drawing. Two of the front boards were then nailed on, concrete was shovelled in to the depth of 12 in., and one transverse tie slid down so as to rest upon the surface of the concrete. The same series of operations was repeated until the work reached the top of the mould.

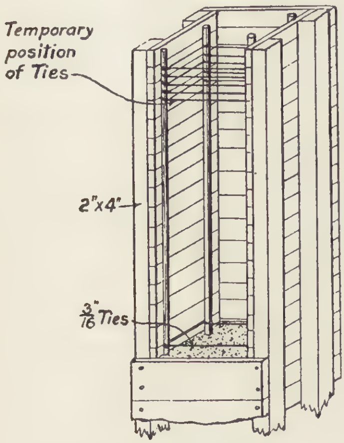


FIG. 80.—Column Mould.

Part of a wall mould is shown in Fig. 81. The sides were formed of 1-in. boards, planed on face and edge, clamped together by horizontal battens, and kept at the proper distance apart by short pieces of board nailed upon the top at suitable intervals. The drawing represents the mould ready for continuing part of a wall already commenced. In building the first course of the wall the mould was set upon the ground, which was rammed and carefully levelled, and the mould was held in position by means of struts on either side. Three days were allowed for the concrete to set, the sides of the mould were then loosened and the mould raised to the position indicated in Fig. 81, where it was held by inserting and tightening the bolts. The sleeves through which the bolts passed were simply of cardboard, this being quite sufficient to prevent the concrete from adhering to the metal, and so obviating any difficulty in the way of removing the bolts.

The wall moulds were made in sections, each 16 ft. long by 3 ft. high, fitting between the wall columns.

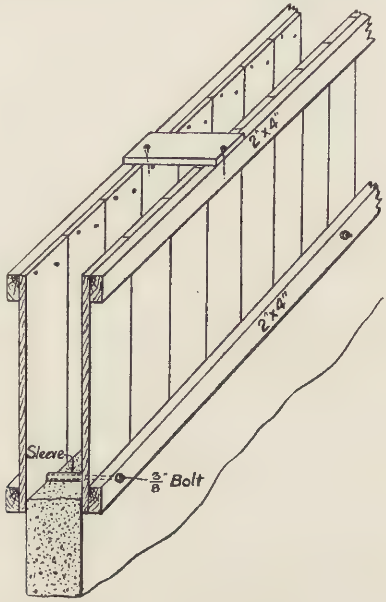


FIG. 81.—Wall Mould.

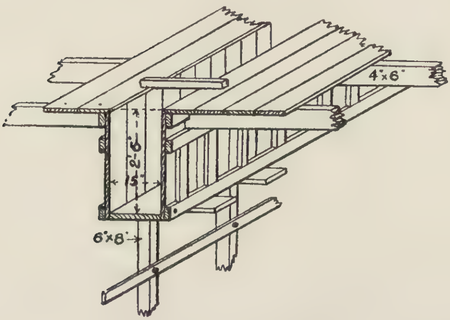


FIG. 82.—Girder Mould.

Fig. 82 shows part of a girder mould and of the boarding for the floor slab. The bottom of the mould consisted of a board or plank, from 1 in. to 3 in. thick, according to circumstances, and attached to the sides by screws passing through longitudinal fillets, as shown in the sketch. The sides were formed of vertical boards 4 in. wide by $\frac{7}{8}$ in. thick, nailed to top and bottom longitudinal fillets, 4 in. wide by 2 in. thick, and also by a third fillet really provided for supporting the 4-in. by 6-in. timbers carrying the floor slab falsework. For the support of the girder moulds, 6-in. by 8-in. struts were wedged beneath short pieces of board acting as caps, as shown in the figure.

After the girders and floor slab had been formed and allowed to harden for about seven days the floor centring was struck by turning the 4-in. by 6-in. timbers on to their sides, permitting the floor boards to follow. In another week the sides of the girder moulds were removed, and the girders remained with the bottom board in place for a further period of three weeks, on the expiration of which the wedges between the struts and the cap boards were knocked away, and the girders were left for another week with the struts in position and ready to afford support in case of failure. The advantages claimed for this method of hardening are that the free access of air facilitates the setting of the concrete, and at the same time offers safeguards in the event of collapse through unsuspected defects of any kind.

One point to which special attention may be drawn is that the reinforcement of the various members was wired and otherwise fixed in the moulds, so that displacement of the bars became practically impossible, or at all events extremely unlikely. Considerable care was taken with the view of ensuring a good bond between new and old work. Generally, it was found that an interval of about twelve hours did not interfere with satisfactory union, but in cases when a longer stoppage of work became necessary the existing surface was well washed and covered with a thin layer of mortar before the resumption of concreting. All junctions were strengthened by the insertion of steel netting of the kind already described.

PRINTING WORKS IN LONDON

72. General Description.—One of the most interesting examples of concrete-steel construction in the metropolis is represented by the extensive premises of which some drawings are here reproduced. Owing to the requirements of the existing Building Act it was decided to build the walls of ordinary brick, but the columns, floors, and the flat portion of the roof are in concrete-steel. The building was erected in accordance with the Hennebique system for Messrs. Hudson & Kearns, at Hatfield Street, London, S.E., from the designs of Mr. F. Matcham, F.R.I.B.A. It measures 210 ft. long by 100 ft. wide, and is about 50 ft. high from the basement floor to the ridge of the roof.

Fig. 83 is a section on the line CD in Fig. 84, and may be termed a plan of the first floor viewed from below. It will suffice to indicate the general arrangement of the columns, floor beams, and walls. Fig. 84 is a transverse section, and Fig. 85 is a longitudinal section along the line AB in Fig. 83.

By Fig. 83 it may be seen that the area of the building is divided into three portions by two main interior walls shaded solid black in the drawing.

73. Columns.—The weight of the floor and roof construction, together with the specified superloads, involve a maximum load of nearly 614,000 lb. per column.

As the columns are only 16 in. square in the basement, this is equivalent to $614{,}000 \div 16^2 = 2{,}400$ lb. per square inch of cross section.

Owing to the magnitude of this load the column construction is one of the most important features of the building under consideration. Fig. 83 indicates the positions of the columns, and Fig. 84 the manner in which these members are extended near the under side of each floor, so as to give ample support and rigidity to the main beams.

The columns measure 11 in. square in the top storey of the building, and 16 in. square between the first floor and the level of the basement floor. Each column is provided with a concrete-steel base (of the type described and

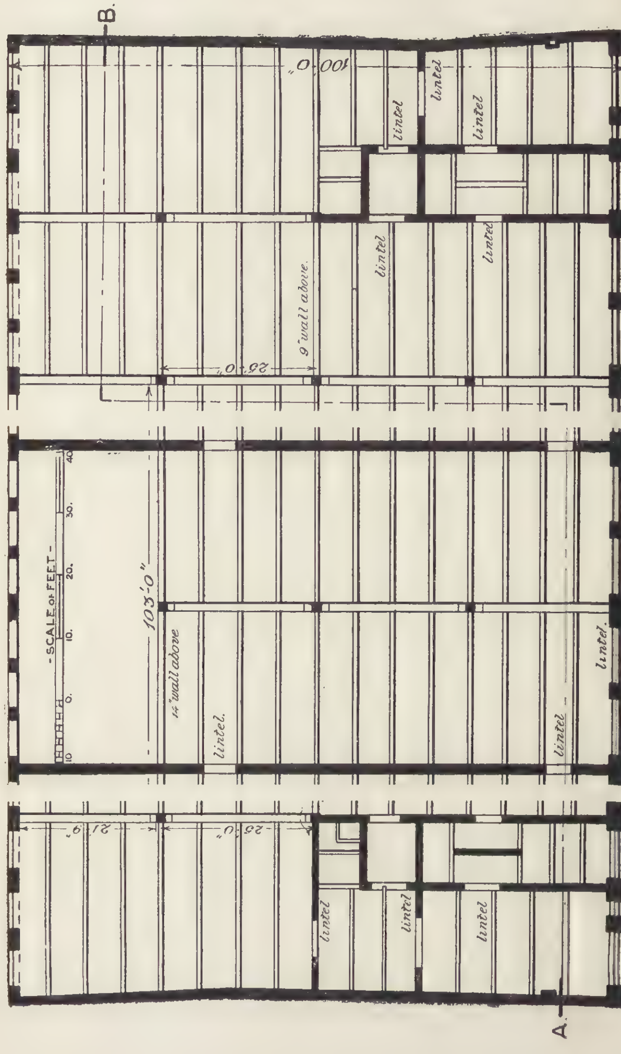


FIG. 83.—Plan of Printing Works in London.

illustrated in Article 57, p. 80), measuring 5 ft. 3 in. square, with a maximum thickness of 15 in. sloping down to the thickness of 9 in. at the edges. These bases distribute the load at the rate of 22,400 lb. per square foot over the masonry foundations upon which they are built.

74. Comparison of Concrete-Steel and Cast-Iron Columns.—It is interesting to compare the sectional area of these concrete-steel columns with that of a cast-iron column suitable for the same calculated load.

Allowing 2 in. as the minimum thickness of fire-resisting material, a 12-in. square cast-iron column will give a finished cross section of 16 in. square, which is the same as that of the lower part of each concrete-steel column.

Let us assume the thickness of metal in the cast-iron column to be 1.5 in., and the length 10 ft. = 120 in., to agree with the distance between the concrete-steel footing and the under side of the ground-floor beams.

Rankine's rule[1] for the breaking load of a column fixed at both ends is

$$P = \frac{f\,S}{1 + \dfrac{l^2}{cr^2}}.$$

As $f = 80{,}000$, $c = 3{,}200$, $l = 120$, we have only to determine the value of r^2, the square of the least radius of gyration.

Denoting the exterior and interior length of side of the hollow square by b and b_1 respectively, we have

$$r^2 = \frac{b^2 + b_1{}^2}{12} = \frac{12^2 + 9^2}{12} = 18.75.$$

Substituting this and the other values in Rankine's formula, we obtain for the breaking load (P) per square inch of the cast-iron column

$$P = \frac{80{,}000 \times 1}{1 + \dfrac{120^2}{3{,}200 \times 18.75}} = \frac{80{,}000}{1.24} = 64{,}520 \text{ lb.}$$

Applying 8 as the factor of safety, we get for the safe load $64{,}520 \div 8 = 8{,}065$ lb. per square inch of metal.

[1] *Structural Iron and Steel*, Whittaker & Co., p. 144.

The sectional area of metal in the hollow cast-iron column is $12^2 - 9^2 = 63$ sq. in., and the safe load is

$$8{,}065 \times 63 = 508{,}095 \text{ lb.}$$

As this calculated load is barely 80 per cent. of that actually carried by the concrete-steel, it would be necessary to increase the thickness of metal to nearly 2 in. for the purpose of ensuring the requisite resistance. But even then the cast-iron column would take up no less room, after the addition of fire-resisting material, than concrete-steel construction.

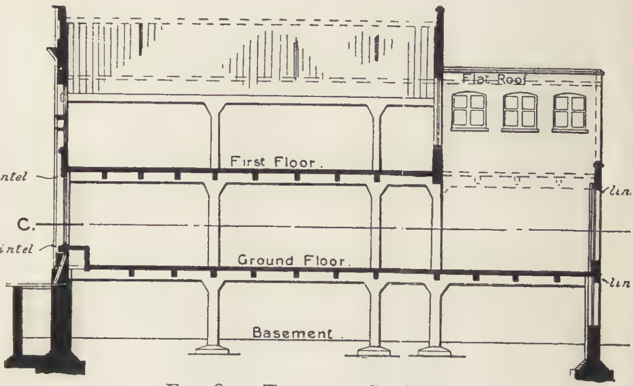


FIG. 84.—Transverse Section.

75. Comparison of Concrete-Steel and Steel Columns.—Let us next make comparison with a steel column from a well-known handbook on steel sections.[1] The column in question measures 12 in. square, the area of metal being 40.59 sq. in., and the safe load with a factor of safety of 4 is given at 234 tons = 524,160 lb., or only about 85 per cent. of the load carried by a 16-in. square concrete-steel column.

Reference to the same handbook shows that, in order to provide for the load of 614,000 lb., it would be necessary to employ a built-up steel column measuring 14 in. square,

[1] *Pocket Companion*, Dorman Long & Co., 1906, pp. 82, 83.

which with a casing of fire-resisting material 2 in. thick would make the total dimensions of the column 18 in. square, which is considerably greater than that of the concrete steel columns actually used.

The foregoing comparisons are sufficient to indicate the advantages of concrete-steel from the standpoints of strength and economy of space.

One other advantage should not be overlooked, namely, the manifest superiority of concrete-steel to any cased form of iron or steel construction in respect of resistance to fire. Large masses of metal merely sheathed in protective material are manifestly far more liable to injury than comparatively thin bars of steel embedded in concrete.

76. Floor Construction. — The main floor beams, 16 in. wide by 19 in. deep, run from front to back of the building, as shown in Fig. 84, except in the case of the two end beams, which commence at the termination of the interior partition walls. The extremities of the other beams are carried by the exterior brick walls, and intermediate support is afforded by the concrete-steel columns, which are spaced 25 ft. apart centre to centre. The clear span of the main beams is 21 ft. 9 in. between the outer walls and the nearest column, and 23 ft. 6 in. between columns.

The distance between the main and other partition walls running from front to back of the building is about 50 ft., and each of these distances is divided by the main floor beams, which together with the brick walls afford support for the secondary floor beams running parallel with the front and back walls. The general dimensions of the joists are 8 in. wide by 15 in. deep, except in places where 9-in. brick walls are supported — the dimensions then being increased to 9 in. wide by 15 in. deep—and in one place, where an upper 14-in. wall is carried, the dimensions are 14 in. wide by 15 in. deep.

The secondary beams are spaced 6 ft. 3 in. apart centre to centre, so that the floor consists of panels, each measuring about 24 ft. 4 in. by 5 ft. 7 in. between supports. These measurements apply only to the larger workshops, as in the rooms formed by the brick partitions at each end of the

building, the secondary beams are spaced somewhat closer together, and measure 5 in. wide by 12 in. deep.

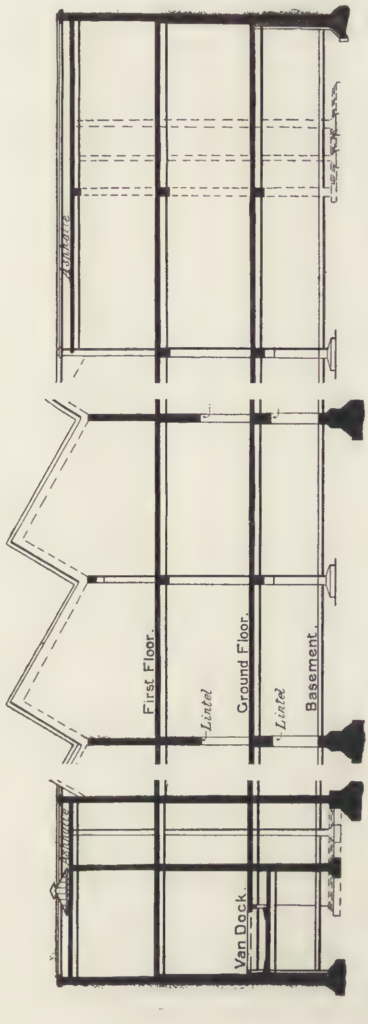


FIG. 85.—Longitudinal Section on line AB (Fig. 83).

All the secondary beams are designed so that continuous girder action may be developed, and, as may be gathered from Fig. 84, the same feature is still more prominent in the case of the main beams.

Fig. 86 is a photographic view showing part of the ground floor, which will serve to indicate the general appearance of the interior.

The ground storey and first storey floor slabs are of concrete-steel 5 in. thick, but the basement floor, being laid on solid earth, is of ordinary concrete.

Both the ground and the first floors were calculated for a superload of 336 lb. per square foot.

77. Walls and Lintels.—The outer walls, being built of brick in conformity with the regulations of the London Building Act, require no comment. The interior partition walls mentioned in Article 72 are also of brick 21 in. thick in the basement, 18 in. thick in the ground storey, and 14 in. thick in the upper storey. Two door openings in each of these walls on the ground and first floors are spanned by concrete-steel lintels 8 in. deep by 21 in. and 18 in. wide respectively.

At either end of the building the interior partition walls, $14\frac{1}{2}$ in. thick up to the first floor, and 9 in. thick above, provide small rooms for use as offices, storerooms, and receiving and despatching-rooms. They also form the walls of lift enclosures and staircases. All the openings in these partition walls are protected by concrete-steel lintels 8 in. deep, and of width equal to the thickness of the brickwork. Other lintels in the front and back outer walls on the ground and first floors measure 9 in. wide by 10 in. deep, with the exception of one, 15 in. wide by 6 in. deep, a short distance above street level at the front of the building.

At points where considerable loads are imposed on the interior brick walls, concrete-steel templates are provided for the purpose of distributing the strain over a sufficient area of masonry to prevent the risk of failure by crushing.

78. Roof.—By Fig. 85 it may be seen that the central part of the roof is of the Yorkshire, or saw-tooth, variety, while at each end there is a flat roof covering an area of

FIG. 86.—View showing part of Ground Floor.

Fig. 87.—View showing part of Roof.

53 ft. by 100 ft. These portions of the roof are formed by 5 in. by 9 in. concrete-steel beams, spaced to correspond with the secondary floor beams below, and covered with a concrete-steel slab 4 in. thick, over which is a layer of asphalt. Fig. 84 shows a transverse section of the flat roof slab in dotted lines, and Fig. 87 shows a portion of the roof near the left-hand side of Fig. 85.

79. Van Docks.—At the front of the works two van docks are provided, so that vehicles may be backed directly into the building. One of these is shown at the left hand of Fig. 85. The floor of the dock consists of concrete-steel beams 5 in. wide by 12 in. deep, with a slab of the same material 5 in. thick, the top of the slab being at the same level as the road surface outside. The second dock is in a similar position at the other end of the building.

A FIVE-STOREY FACTORY BUILDING IN PHILADELPHIA

80. General Description.—This building was recently erected in Philadelphia for use as a machine shop. It has a frontage of 120 ft., and measures 100 ft. from front to back. Fig. 88 is a section showing the five storeys in the front portion, and the single storey extension with a Yorkshire roof at the back. With the exception of the outer walls, which are of brick with stone and terra-cotta facings, the entire structure is of concrete-steel. A general idea of the building will be obtained by examination of the section, where the outlines of the main structural features are indicated.

81. Columns, Floors, and Roofs.—In the front portion the interior columns are disposed in two parallel rows 18 ft. apart centre to centre, and, as indicated in the figure, the dimensions of the columns vary from 2 ft. 2 in. square in the basement to 8 in. square in the top storey. The columns of each row are connected by concrete-steel main beams, of which the cross section measures 10 in. wide by 14 in. deep on each floor, these beams being connected by secondary beams measuring 6 in. wide by 12 in. deep. All the floor slabs between the main beams and joists are of concrete-steel.

Fig. 89 is an isometric drawing of a portion of the work, including part of one column *a*, of two girders *bb*, of three joists *ccc*, and of the floor slab *dddd*. This figure shows the appearance presented by the under side of the first floor.

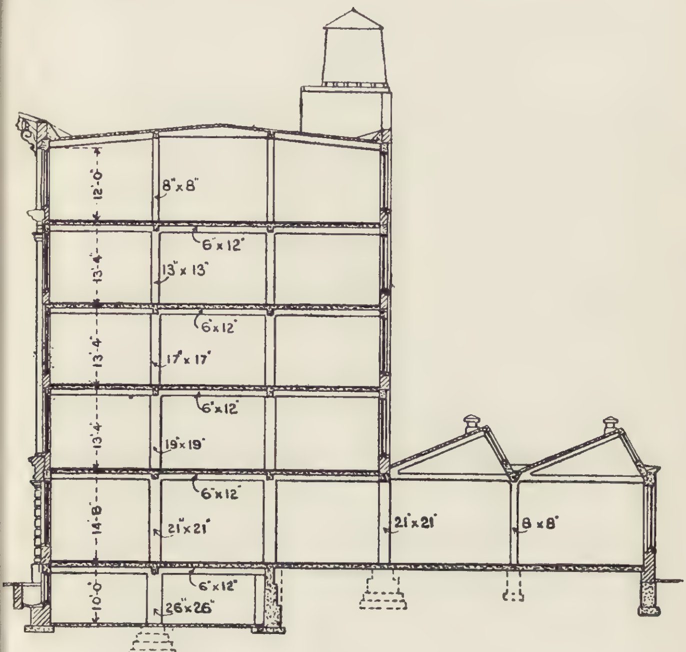


FIG. 88.—Factory Building in Philadelphia.

At the back of the front portion of the building the outer wall is supported by a row of concrete-steel columns, with a cross section of 21 in. square, connected with concrete-steel girders.

The floor slab between the girders and joists is 3 in. thick, and covered with $1\frac{1}{4}$-in. floor boards of maple laid on 2-in. sleepers 17 in. apart centre to centre. This method

of floor covering applies to all the upper storeys of the building, the surfaces of the basement and ground floors being finished in cement.

The roof is formed of concrete-steel beams, with a covering of slag laid on concrete, and at the back of the main block there is a small tower containing a water tank.

The interior columns, beams, and roof of the back portion are also of reinforced concrete, the roof covering

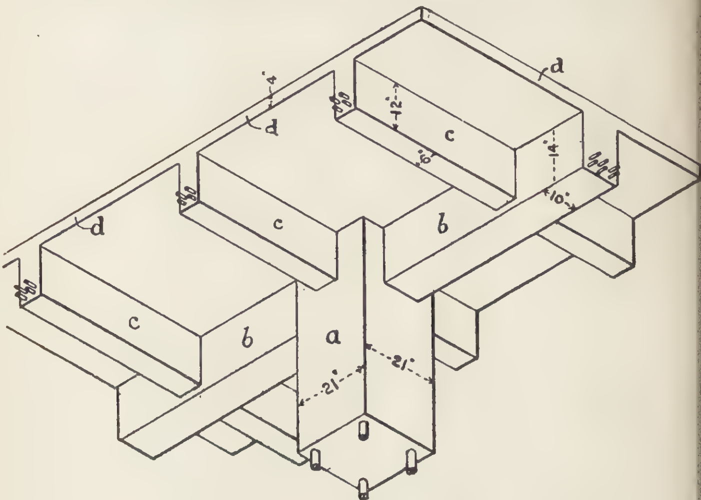


FIG. 89.—Perspective of Column and Floor Construction.

of the longer slopes being of slag laid on concrete and supported by concrete-steel principals, the shorter slopes being filled with skylights between the principals.

82. Reinforcement of Columns and Beams.—The details of the reinforcement in the columns and beams are shown in Fig. 90, wherein are represented parts of the columns above and below the first floor and a portion of the first floor itself. In the columns the reinforcement consists of four vertical bars passed through holes in sets of four flat bars spaced about 16 in. apart in a vertical

direction. In the 6-in. by 12-in. beam, or joist, the reinforcement comprises two horizontal bars in the tension area, these bars running through the columns so as to form continuous reinforcement; and two bent bars, providing for tension in the upper portion of the cross section near the ends and in the lower portion of the section at the middle of the beam. There are also two horizontal bars near the upper surface of the beam, and a series of stirrups for assisting the concrete to resist shearing stresses. The reinforcement of the main beams is not fully shown in the drawing, as the section of these is taken at the middle of the span, where the lower horizontal bars and the bent bars meet.

83. Column, Beam, and Roof Moulds.—Fig. 91 is an isometric drawing of the moulds and centring employed for

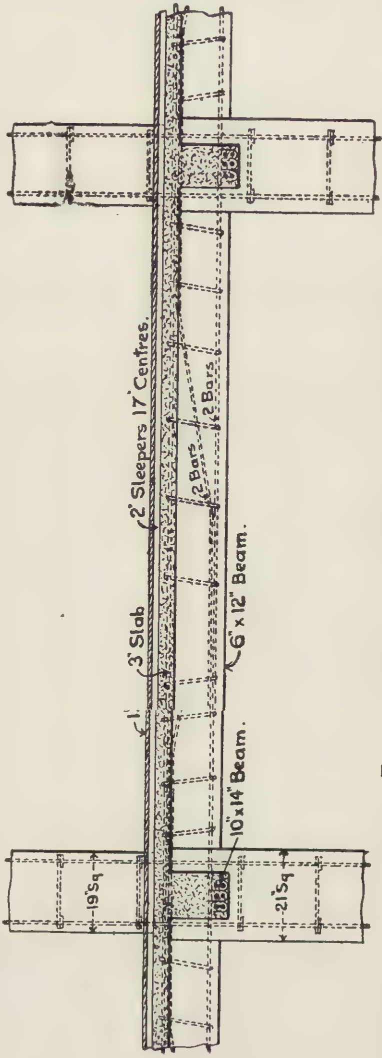


FIG. 90.—Details of Columns, Beams, and Floor Slab.

forming the concrete of the columns, girders, joists, and floor slab. The upper end of the column mould *a* is framed to the moulds for the girders *bb* and for the joists *ccc*, and the floor centring *ddd* is nailed down to the side boards of the moulds. In addition to the support afforded by the column moulds, the beam moulds are supported by means of struts, as *ee*, placed at suitable intervals.

In depositing the concrete, the column moulds were filled up to floor level, all the other moulds were then filled, and the floor centring was covered with the least possible

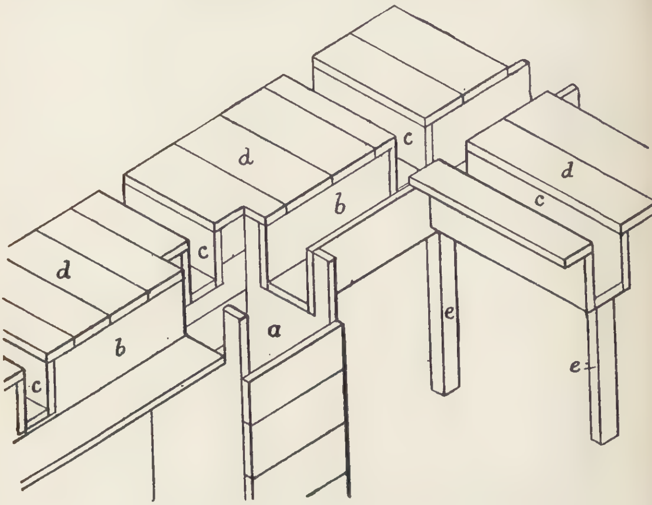


Fig. 91.—Column, Beam, and Floor Slab Moulds.

delay, so as to avoid any discontinuity between the different portions of the work. The joints of the column reinforcement were made well above the different floors, and numerous bolts were set in the floor slab for the purpose of fixing the timber sleepers for the maple boarding. Other bolts were built into the lower part of the girders for the attachment of hangers for the shafting to be used for

running machinery. These bolts were passed through holes in the bottom boards of the various moulds.

Fig. 92 is a perspective drawing of the moulds used for the Yorkshire roof extension at the back of the building. It will be seen from this illustration that the entire framework is of concrete. The drawing here reproduced has

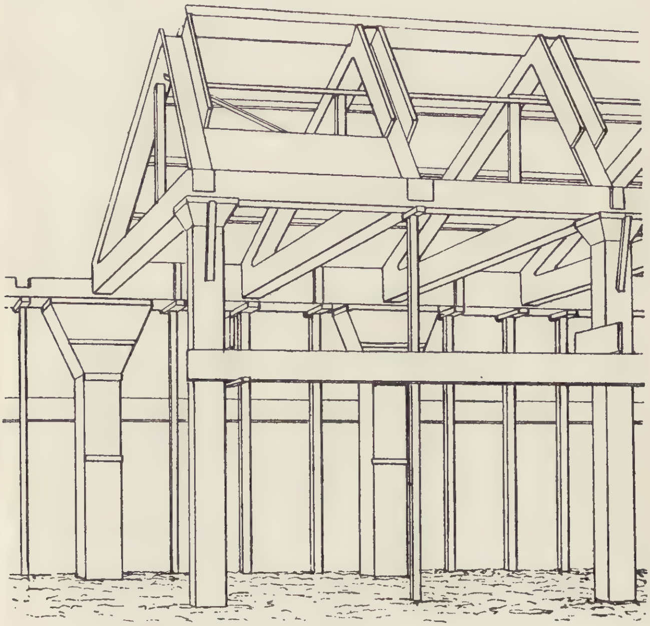


FIG. 92.—Moulds for Columns, Beams, and Roof.

been made from a photographic view taken at ground level before the moulds were completed by the addition of the boarding upon which the concrete roof covering was deposited. The figure includes a portion of the forms used for the construction of one section of the single-storey extension.

In the background will be observed the moulds for the

columns to which are transmitted the loads carried by the back wall of the five-storey building and the weight of the wall itself. The columns in question are provided with bracketing for the more adequate support of the continuous main girder running along the whole width of the structure.

In addition to the skylights in the shorter slope of the roof, several openings for ventilators were provided in the longer slope. These openings were formed by placing core boxes upon the roof boarding and leaving the boards inside the core free from concrete when the roof slab was formed.

Owing to the uniformity of construction of the different floors, the same moulds could be used in succession for all the storeys, and the only modification necessary was the diminution of the cross section of the column moulds in accordance with the dimensions shown in Fig. 88.

Similarly, only one set of moulds was required for the single-storey extension, as this was erected in sections of uniform area, and the temporary framework was moved from section to section until the whole of the structure was completed.

Thus the cost of the forms was reduced to a minimum, so that it bore but a very small proportion to the total value of the work.

84. Floor Loads. — Being designed for the use of machinery, the ground floor was proportioned for a uniformly distributed load of 300 lb. per sq. ft., the first floor for a similar load of 200 lb. per sq. ft., and each of the floors above for a load of 150 lb. per sq. ft.

BUSINESS PREMISES IN SOUTHAMPTON

85. General Description.—The building illustrated in the accompanying series of drawings was designed by Messrs. Poole & Sons, in accordance with the Hennebique system, and the concrete-steel construction was executed under the direction of Mr. L. G. Mouchel, M.Soc.C.E. (France). This structure was designed to serve the purpose of a drapery business, and is situated at the corner of East Street and Strand, Southampton. Fig. 93 is a general view

of the premises, the outer walls of which were built in brick, with the addition of masonry in the form of cornices, mullions, and balustrades, but the whole of the interior work — including columns, column foundations, floors, partition walls, and staircases, as well as the roof—is of concrete-steel, and columns of the same material are also incorporated in the outer walls.

Fig. 93.—Business Premises in Southampton.

The principal façade, which is on East Street, has a length of 40 ft., the other façade on the Strand having a length of 47 ft. Fig. 94 is a section from front to back of the building, which includes four storeys exclusive of the basement, the height being 64 ft. from foundation level to the top of the flat roof and 69 ft. to the top of the mansard roof. Above the latter is a storage tank of circular form

constructed in concrete-steel, the tank having a mean diameter of 5 ft. 6 in. and a depth of 2 ft. 9 in., outside measurements.

On the Strand frontage two openings are provided, as shown in Fig. 95, giving access to the basement, the larger opening for merchandise and the smaller one for coals. It is unnecessary to devote space to a description of the exterior walls of the building, the disposition and relative thickness of which are made sufficiently clear by the basement plan (Fig. 95), and on the ground floor plan (Fig. 96). We may mention in passing that the footings consist of concrete, and are 5 ft. wide by 2 ft. deep. The broken lines along the street frontages and the party wall lines in Fig. 95 indicate the outside edge of the foundation course, of which two sections will be observed in Fig. 94.

86. Columns.—Fig. 95 shows the arrangement of the interior columns and the areas of their bases. Typical outline sections of the bases are included in Fig. 94, by which it may be seen that the slope of the upper surface is varied according to requirements, but the uniform thickness of 12 in. has been adopted as a minimum. In every case the lower surface is at a level of 4 ft. 6 in. below the basement floor.

For the five thin columns incorporated in the brick walls of the building (see Fig. 95) the foundations measure 6 ft. square by 1 ft. 9 in. deep at the centre, tapering down to 12 in. deep at the edges. Of the five interior foundations three measure 6 ft. square by 1 ft. 6 in., and 12 in. deep at the centre and edges respectively; one, which has to afford support for a large and a small column, has a length of 7 ft. 8 in., a width of 6 ft., and thicknesses of 1 ft. 3 in. and 12 in.; and the last, which supports a small column near the back of the premises, measures 4 ft. square, and has maximum and minimum thicknesses of 1 ft. 6 in. and 12 in. respectively.

The five columns in the exterior walls are necessary for the reinforcement of the brickwork, so as to afford adequate support for the concrete-steel lintels carrying the walls and masonry over the window and door openings on the ground floor. It was necessary to make these columns of the

smallest possible cross-sectional area, and with a minimum thickness. In consequence of this requirement they were designed so as to measure only 7 in. thick by 18 in. wide.

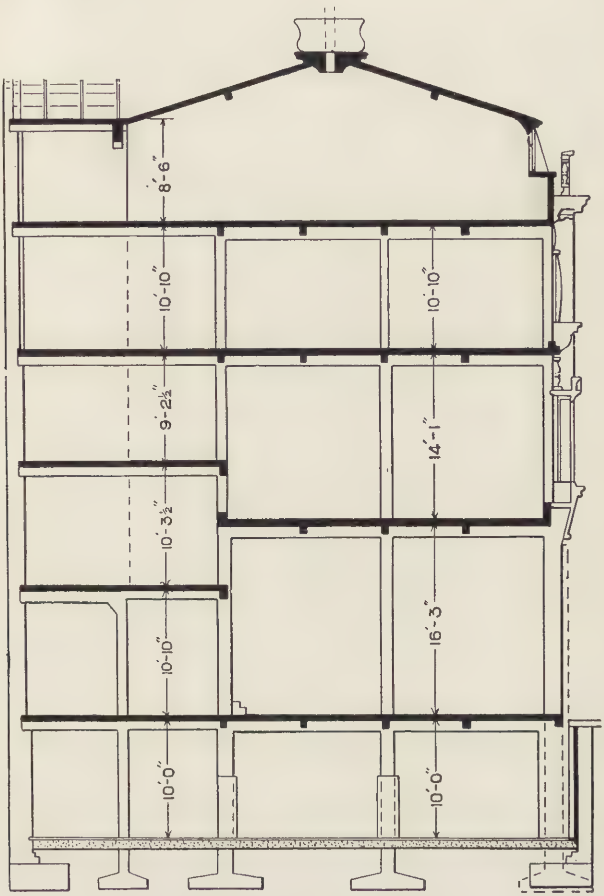


FIG. 94.—Longitudinal Section.

The calculated load upon each of these slender columns is about 116,500 lb., and as the cross-sectional area is only $7 \times 18 = 126$ sq. in., the load is approximately 925 lb. per sq. in. The height of the basement is 10 ft. = 120 in.

87. Comparison of Concrete-Steel and Cast-Iron Columns.—By calculating the safe load for a hollow cast-iron column with sides $\frac{1}{2}$ in. thick of equal length and cross section, equal to those of the concrete-steel columns, it will be found that the load actually carried by each of the latter does not compare at all unfavourably, especially when the relative weights of the materials are taken into account. Of course, much depends upon the basis of the calculation and the factor of safety allowed.

If it be assumed that the ends of the cast-iron column are not rigidly fixed the computed result will be much smaller than that calculated upon the assumption that the ends are so fixed.

Rankine's formulæ[1] for the breaking load of columns are :—

$$P = \frac{fS}{1 + \frac{l^2}{cr^2}}$$

where both ends are rigidly fixed, and

$$P = \frac{fS}{1 + \frac{16l^2}{9cr^2}}$$

where one end is rigidly fixed and the other is rounded or jointed. The value of Rankine's coefficients[1] for cast iron are—

$$f = 80{,}000 \text{ lb.}, \quad c = 3{,}200.$$

The value of r^2, the square of the least radius of gyration, calculated as in Article 74, is—

$$r^2 = \frac{b^2 + b_1^2}{12} = \frac{7^2 + 6^2}{12} = 7.$$

[1] *Structural Iron and Steel*, Whittaker & Co., p. 144.

and as in the present example l = 120 in., the breaking load per square inch for both ends rigidly fixed would be—

$$P = \frac{80{,}000 \times 1}{1 + \dfrac{120^2}{3{,}200 \times 7}} = \frac{80{,}000}{1.64} = 48{,}800 \text{ lb.}$$

Adopting 8 as the factor of safety, the safe load becomes 48,800 ÷ 8 = 6,100 lb. per sq. in.

As the area of metal in the cast-iron column would be (7 × 18) − (6 × 17) = 24 sq. in., the safe load for the column is 6,100 × 24 = 146,400 lb., or about 25 per cent. more than that for the concrete-steel column. But calculating the breaking load in accordance with the safer assumption that only one end is rigidly fixed, we get—

$$P = \frac{80{,}000 \times 1}{1 + \dfrac{16 \times 120^2}{9 \times 3{,}200 \times 7}} = \frac{80{,}000}{2.14} = 37{,}383 \text{ lb.}$$

Then with 8 as the factor of safety, the safe load per square inch is 37,383 ÷ 8 = 4,673 lb., and as the area of metal is 24 in., the safe load for the column is 4,673 × 24 = 112,152 lb., or about $3\frac{3}{4}$ per cent. less than that for the concrete-steel column.

The four principal independent interior columns of the building at Southampton are of square section, as shown in Figs. 95 and 96. The dimensions commence at 14 in. square in the basement, and diminish to 8 in. square at the third floor. The total load carried by each column is about 200,000 lb., equal to 1,020 lb. per sq. in. of sectional area.

It would be interesting to make calculations for comparing concrete-steel with cast iron in the case of these columns. This time, however, we will make an approximate computation by a table of hollow cast-iron columns.[1]

The height of the 14-in.-square portion of the concrete-steel columns is 120 in. Allowing 2 in. on each side for fire protection, we have 10 in. square as the available dimensions of a cast-iron column, in which length ÷

[1] *Structural Iron and Steel*, Whittaker & Co., p. 151.

diameter $= 120 \div 10 = 12$. The value for this ratio by the table is 62,110 lb. for a square column, this being the breaking load per sq. in. Dividing by 8 to give the same factor of safety as before, we get—

$$62{,}110 \div 8 = 7{,}763.$$

This is the safe load per square inch, and the requisite of metal is—

$$200{,}000 \div 7{,}763 = 26 \text{ sq. in.}$$

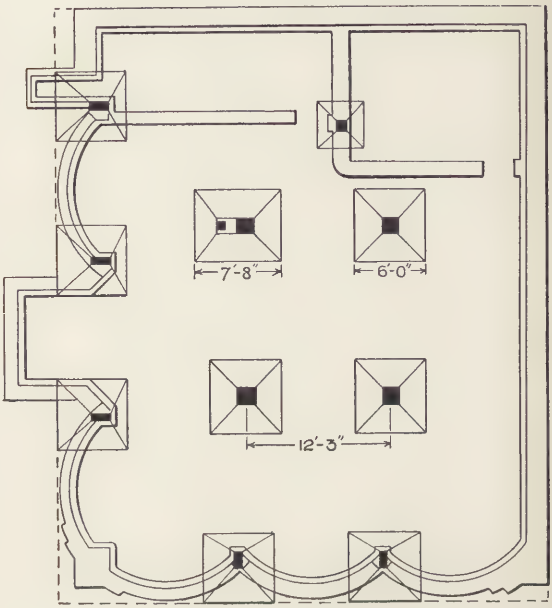


FIG. 95.—Basement Plan.

Thus the area of the hollow inside the column would be $10^2 - 26 = 74$ sq. in., and the thickness of metal would be 0.7; say $\frac{3}{4}$ inch.

Therefore the 14-in. square concrete-steel column is equal in strength to a 10-in. square hollow cast-iron column with sides about $\frac{3}{4}$ in. thick, and the final dimensions of the latter after the addition of fire-resisting material would be very similar to those of the concrete-steel construction.

An additional element of strength is afforded in the case of concrete-steel by the secure connection of the longitudinal reinforcement with the concrete of the main beams, and by the monolithic character of the entire work.

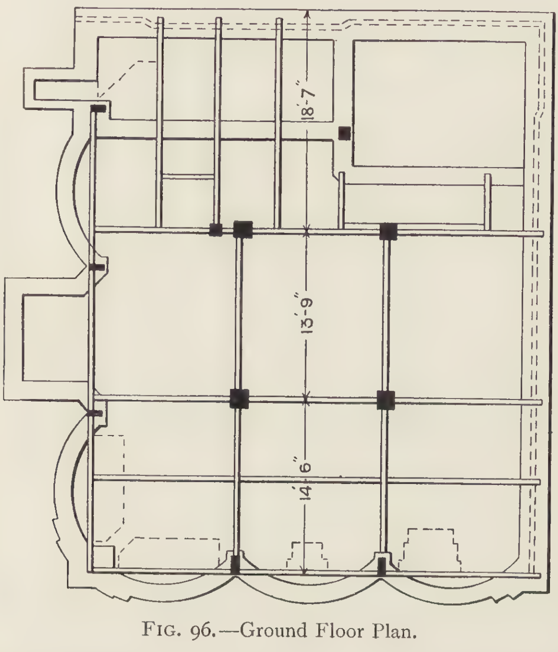


FIG. 96.—Ground Floor Plan.

88. Floor and Roof Construction.—Fig. 96 is a plan of the ground floor showing the arrangement of the beams.

The span of the main beams on the ground floor is 13

ft., the cross section of these beams being 9 in. deep by 7 in. wide, and the thickness of the intervening floor slab is 4 in.

On the first floor the span of the main beams is 13 ft., the section of the beams 9 in. deep by 8 in. wide, and the thickness of the floor slab $4\frac{1}{2}$ in.

On the first and second mezzanine floors the floor slab is 4 in. thick. At the first mezzanine floor there are two concrete-steel lintels 15 in. deep by 14 in. wide, which carry one of the 14-in. exterior brick walls from this level up to the roof, as well as part of the roof load and the weight of some of the other floors. On the second mezzanine floor part of the floor system is designed on the cantilever principle, being carried on a beam which in turn is a cantilever supported by some of the other main beams.

For the second and third floors the main beams measure 9 in. deep by 7 in. wide, and the floor slab $4\frac{1}{2}$ in. thick.

The roof is constructed entirely in concrete-steel without intermediate support, and consists of concrete-steel principals with a span of 37 ft., connected by a roof slab of the same material.

89. Floor Loads.—The following were the calculated superloads for the different floors:—

Ground floor . . .	168 lb. per sq. ft.	
First floor	280	,,
Mezzanine first floor . .	112	,,
Mezzanine second floor .	112	,,
Second floor	224	,,
Third floor	224	,,

CHAPTER VI

Isolation Pavilions and Roof of Diphtheria Block in a Hospital, Paris—Electric Tramway Depôt, Paris—Maison de Rapport, Paris—L'Eglise de S. Jean de Montmartre, Paris

Note.—The buildings described and illustrated in this chapter represent a type of reinforced concrete and brick construction which differs considerably from other methods of reinforced construction. Therefore it has been thought desirable to collect all the examples in one chapter, notwithstanding the fact that they represent varied classes of buildings.

ISOLATION PAVILIONS

90. General Description.—The pavilions of which particulars are given below were designed by Monsieur Belouet, Architecte de l'Assistance Publique, for l'Hôpital des Enfants Malades, Paris, and the reinforced construction was executed in accordance with the Cottancin system.

At the time when they were designed one diphtheria pavilion was already in existence at the hospital, this building being of ordinary construction with walls carried down to a masonry foundation. The floor was of brick with steel joists, and the walls of the wards of brick lined with timber.

This pavilion was much criticised by Dr. F. Roux, the medical superintendent, who considered it very desirable to have a floor under which air could circulate freely, because, owing to the imprisonment of effluvia given off from the soil, the hygienic conditions of the building were far from satisfactory. He criticised also the door and window frames, in which microbes collected and multiplied, and the space existing between the roof covering and the

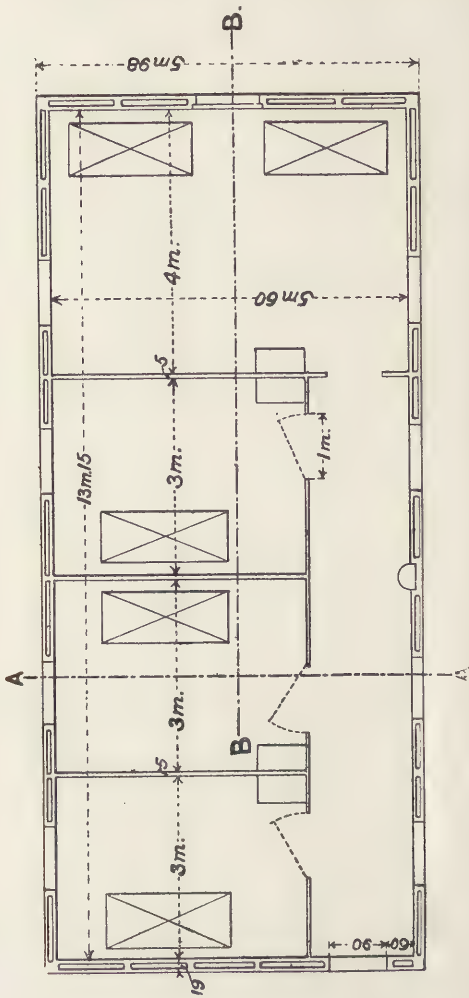


FIG. 97.—Isolation Pavilion, l'Hôpital des Enfants Malades, Paris (Ground Plan).

ceiling, which he found in the same undesirable condition as the space below the floor. Consequently, when it became necessary to build two additional isolation pavilions for the treatment of diphtheria Dr. Roux suggested the main features of the design which was subsequently prepared by the architect.

Two new pavilions have now been erected identical in size and arrangement, each having an internal area of 13.15 metres by 5.60 metres, as shown in Fig. 97, which is a typical ground plan.

The whole of the building is so reinforced in all details, and so tied together by steel, that it really constitutes a great tubular beam, 13.55 metres long by 5.98 metres wide by 4.59 metres high, formed by the floor, the walls, and the roof. This being so, it was unnecessary to employ lintels for the windows and doors, as the openings could be formed much as they might be in steel plate.

91. **Foundations.**—The pavilions are built upon ten reinforced brick piers 33 centimetres square by 2 metres high, so that the floor of the pavilion is 2 metres above the ground level (see Figs. 98 and 99). The piers, situated at the four corners of the pavilion and at three intermediate points in each side wall, are founded upon six caissons, or rectangular brick chambers closed at the top and open at the bottom. These caissons, measuring 1 metre square by 0.6 metre deep, are sunk in the ground, which is of bad quality on the site of the hospital, consisting of made earth overlying the débris of the old quarries of Paris. The support afforded by the caissons was so satisfactory that no deformation of the building took place, the whole structure resting as a great beam upon the ten piers. The slab forming the top of each caisson is of concrete-steel, and situated 10 centimetres below the normal ground level. To prevent the caissons from being laid bare by rain, sills of reinforced brick were formed round the piers built upon the reinforced concrete slabs.

92. **Floor.**—Upon the ten piers was built the floor, which consists of a concrete-steel slab, with beams of concrete-steel 15 centimetres deep by 5 centimetres wide, having metal furring for the plaster ceiling, which forms

an enclosed air space under the floor slab (see Figs. 98 and 99).

93. **Walls.**—Along each outer edge of the floor two tiers of reinforced brickwork 6 centimetres thick were built 7 centimetres apart forming hollow outer walls, the interior air spaces being in communication with the air space of the floor.

94. **Roof.**—As shown in Fig. 98, the roof of the building is formed by hollow arches, these being of concrete-steel built monolithic with the walls.

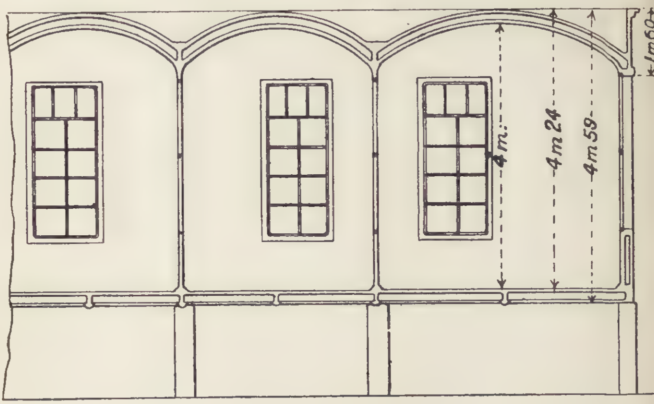


Fig. 98.—Section BB.

95. **Air Circulation System.**—A duct in reinforced brick (Fig. 99) was formed below the floor for the conveyance of warmed air from a calorifier. This warmed air first diffuses itself in the air space of the floor, and then ascends by way of the hollow walls to the spaces in the roof, where suitable outlets are provided for its discharge. Thus it will be seen that the pavilions are constructed with a double casing represented by floors, walls, and roof respectively, and that, between the inner and outer surfaces, warm air circulates which cannot enter any of the wards, and consequently cannot vitiate the air. Further, as the

supporting piers are not connected by walls, there can be no collection of unwholesome emanations from the soil.

96. Interior Fittings.—The isolating partitions shown in Figs. 98 and 99 are of reinforced brick for a height of about 1.50 metres above the floor, and above that level they consist of glazed partitions. This arrangement applies also to the corridor. Thus the attendants are able to supervise the five beds in each pavilion as if no partitions existed between the wards.

The interior doors are in enamelled iron covered inside with plaster appropriately coloured by antiseptic paint, and

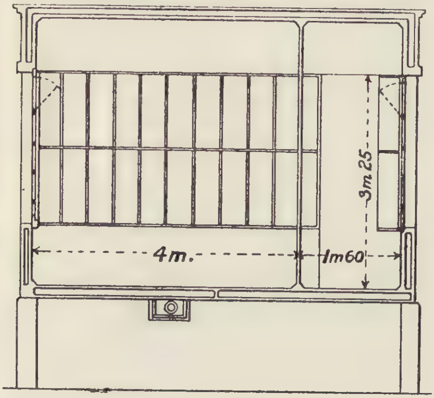


FIG. 99.—Section AA.

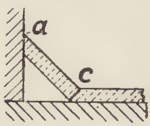


FIG. 100.

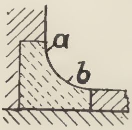


FIG. 100*a*.

a plain fanlight is provided over each door. The interior wall surfaces are rendered in cement and covered with glazed tiles. The angle between the walls and the floor is filled by a course of flat tiles set at an angle of 45 degrees, as shown in Fig. 100.

This arrangement was adopted in preference to the use of tiles or blocks with a concave surface, because water used in washing the walls flows rapidly from *a* to *c*, and is carried away by the washing of the floor. On the other hand, with blocks as in Fig. 100*a*, water trickling down the walls loses velocity between *a* and *b*, tending to lodge near the latter point.

The method of glazing originally proposed being considered unsatisfactory for the reason that it would encourage the collection of dust and organisms, the architect adopted the arrangement proposed by M. Cottancin, of which characteristic details are represented in Figs. 101 to 103.

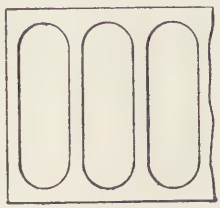
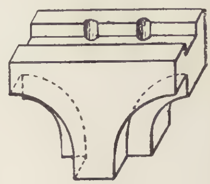


Fig. 101.



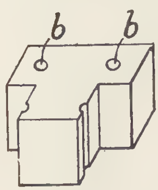


Fig. 102.

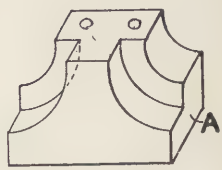


Fig. 102*a*.

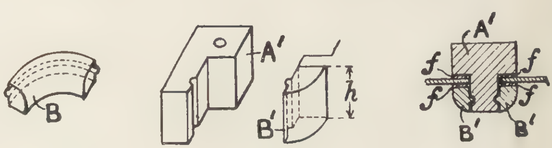


Fig. 103.
Details of Glazing System.

All the parts here illustrated are of moulded glass, connected by means of steel rods, forming a frame, like that shown in Fig. 101, held in position by means of horizontal steel bars.

The block illustrated in Fig. 102 is one forming part of a vertical sash bar. Through the holes *bb*, steel rods of 4 millimetres diameter are passed, being fixed by cement

grout. The rebates and vertical grooves are to provide for fixing the glass on either side of the block as explained below.

The blocks forming the top and bottom parts of the frame are illustrated by the two drawings in Fig. 102*a*. In addition to the vertical holes in these a groove is moulded in the top surface of the upper block, and a similar groove in the bottom surface of the lower block, these grooves being intended to receive the horizontal steel bars by which the complete frame is held in position. The curved rebates in these blocks are made with grooves (not shown in the drawing) to provide for fixing curved segments (as B, Fig. 103), which hold the glass in position. In Fig. 103, A′ is a reversible half block for the bar at either end of the frame, and B′ a block having a projecting rib which fits into the groove in A′ for holding the glass in place. The remaining drawing in Fig. 103 is a horizontal section through the vertical bar of a window frame, A′ being the fixed bar, B′B′ the side fillets, and *ff*, *ff*, strips of antiseptic felt on either side of the sheets of glass. The fillets B′B′ are fixed by means of putty or cement of any suitable composition. This type of window frame obviates the use of exposed metal, and, the construction being entirely of reinforced glass, complies with all hygienic requirements.

ROOF OF DIPHTHERIA BLOCK

97. Problem for Solution.—The roof covering of the main building for the treatment of diphtheria at the same hospital presents a very interesting study from the standpoint of construction as well as of hygiene.

In the first place, it may be mentioned that the problem for solution was to cover a ward measuring 28 metres long by 12 metres wide by a roof resting upon ordinary brick walls, 22 centimetres thick, without providing any intermediate support within the rectangle of 28 metres by 12 metres = 336 square metres area. Consequently, it was necessary that the weight of the roof system should be carried entirely by the walls.

For a roof designed in the ordinary manner the employ-

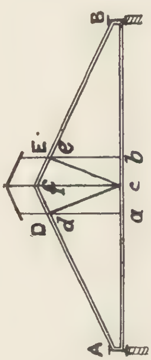


FIG. 104.

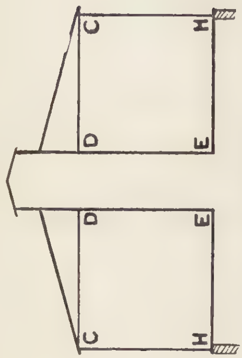


FIG. 105.

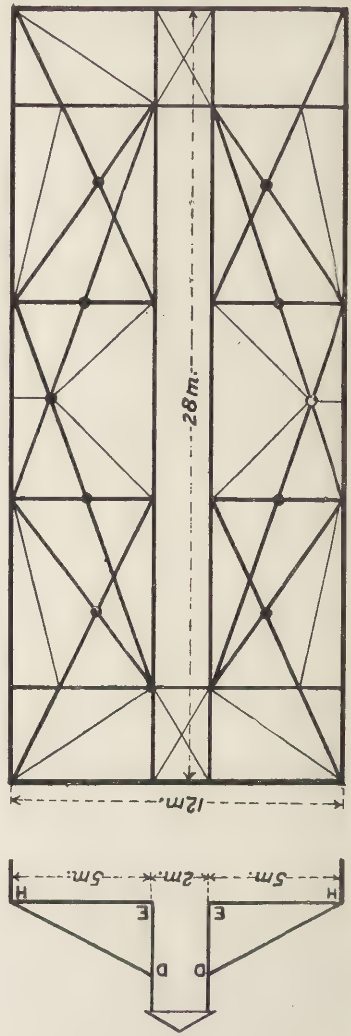


FIG. 106.—Roof of Diphtheria Block, l'Hôpital des Enfants Malades.

ment of wall plates capable of acting as beams would be imperative for the purpose of distributing the loads transmitted by the roof trusses in such manner as to avoid overstraining the brickwork.

The architect proposed to adopt wall plates consisting of built-up steel girders, and carrying trusses spaced 4 metres apart.

Fig. 104 illustrates the proposed method of construction, which comprises two wall plates A and B and a truss carrying a ceiling A*acb*B, and two partitions *ad*, *be* for the passage of light through openings in the ceiling from the lantern above. The medical staff, however, required the suppression of the members *ab*, *cd*, *ce*, *cf*, *df*, *fe*, in order to avoid the collection of microbes in this part of the structure. This demand involved the conversion of the side partitions *ad* and *be* into two beams each 28 metres long. Taking into account the great span and the thrust of the principals AD and BE at the points *d* and *e*, colossal proportions would have been necessary for the beams *ad* and *be*, the weight of which would have been sufficient to crush the thin end walls of the building.

An alternative proposal was then made to construct in steel two tubular beams, CDEH, CDEH (Fig. 105), to carry the roof proper. This project was found to be impracticable because of its ungainly proportions and heavy cost.

98. Solution of the Problem.—The elegant solution proposed by M. Cottancin and accepted by the hospital authorities was to make two triangular tubular beams connected at the ends by ribs and at the top by a lantern as shown in Fig. 106. The sides ED, ED are of reinforced brick 7 centimetres thick; the horizontal members HE, HE are of reinforced concrete 5 centimetres thick, with stiffening ribs of the same material 20 centimetres deep by 5 centimetres wide, following the triangulation in the plan; and the inclined members HD, HD consist of a triangulation of reinforced concrete ribs 20 centimetres deep by 5 centimetres wide, in the vertical plane of the triangulation of the members HE, HE. At five points in each main triangular beam vertical members of concrete-

steel reunite the systems HE, HE and HD, HD. The roof surfaces are formed by slabs of concrete-steel connecting the ribs of systems HD, HD covered with tiles for the sake of appearance, and the ceilings consist of thinner slabs connecting the ribs of the systems HE, HE. This roof complied entirely with the hygienic requirements of the medical staff, and, virtually constituting a single monolithic beam with an opening in the middle, is a striking example of the adaptability of concrete-steel to building construction, as well as a testimony to the ingenuity of its designer.

ELECTRIC TRAMWAY DEPÔT, PARIS

99. General Description.—This building is situated in the Rue de Lagny, near the Porte de Vincennes, Paris. It was designed for the Compagnie Générale des Omnibus by M. P. Cottancin, M.Soc.C.E. (France), and was erected by him under the supervision of M. Monmerque, Ingenieur-en-chef des Ponts et Chaussées, chief engineer to the company. The depôt was established to accommodate an installation of accumulators for the storage of electricity in connection with the tramway services of the company, and to provide local facilities for the repair of cars. The building comprises two storeys, and adjoins the power house where electricity is generated.

As will be seen by reference to Fig. 107, the ground floor accommodation includes a repair shop measuring 16.09 metres long by 8.65 metres wide, at one end of which is a small brass foundry measuring 6.00 metres by 5.00 metres, and at the other end a coal store measuring 9.14 metres long by 7.60 metres wide. On the first floor there are two accumulator-rooms (see Figs. 108 and 109), the dimensions of which are 16.09 metres by 8.65 metres, and 9.14 metres by 7.60 metres respectively. The entire building is covered by a flat roof of concrete-steel.

In order to ensure easy access to the coal store for vans, it was decided to leave an opening 6.72 metres wide in the wall facing the yard (*ab*, Fig. 107). Similarly, an opening of maximum width was required along the outer

side of the workshop to permit easy access for tramcars, and it was necessary that the opening should be without any intermediate support.

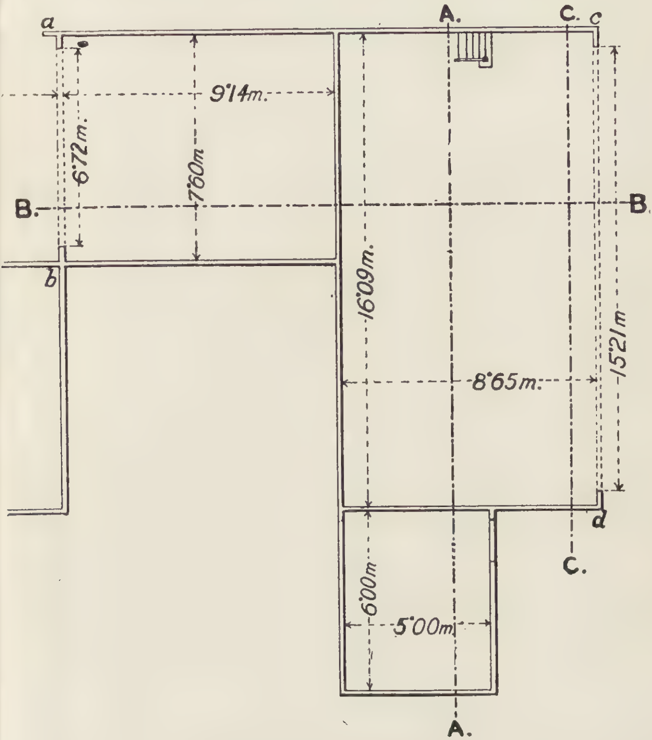


FIG. 107.—Electric Tramway Depôt, Paris (Plan).

Owing chiefly to the weight of the storage batteries, the calculated load for the upper floor was determined at 3,500 kilogrammes per square metre (717.5 lb. per square foot).

Hence the chief problem presented for solution was the

construction of two girders, one 6.72 metres long over the open side of the coal store, and the other 15.21 metres

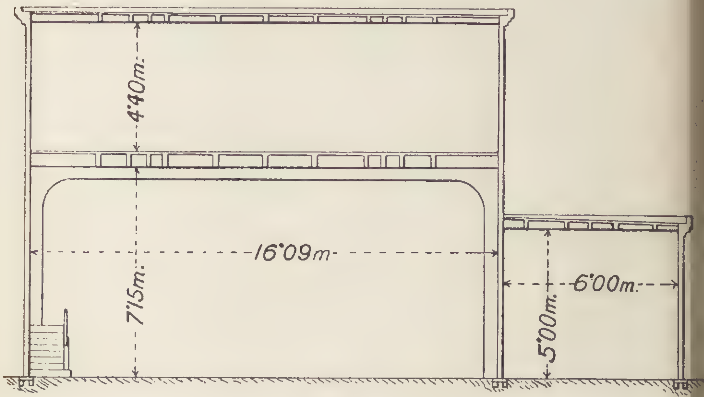


FIG. 108.—Section AA (Fig. 107).

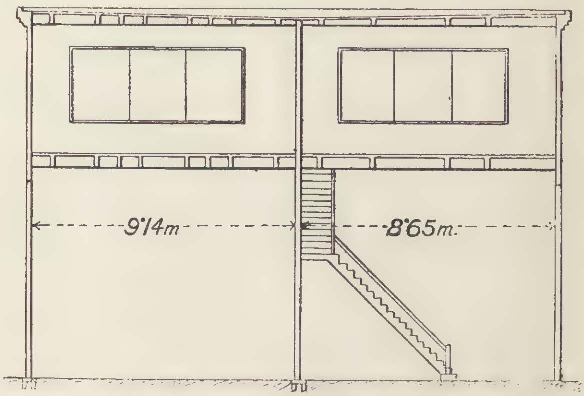


FIG. 109.—Section BB (Fig. 107).

long over the open side of the repairing shop, each of these members having to carry one side of the upper storey and roof, as well as a considerable portion of the floor load.

In view of the unusually high floor load it is evident that ordinary methods of construction would have required girders of very large proportions, and of such weight as to require exceptionally massive supports. The 15.21-metre opening would probably have necessitated a box or plate girder with a section measuring about 36 in. deep by 18 in. wide, and the 6.72-metre opening a similar girder with a cross section of about 24 in. deep by 14 in. wide.

In order to obviate the employment of heavy steel beams the building was constructed in reinforced concrete and brick on the Cottancin system, and designed in such a manner that it is capable of withstanding all the loads coming upon it, without the aid of any special members for carrying the loads over the openings in the outer walls. In fact, the wall over each opening itself forms the girder, as will be realised after consideration of the details given below.

100. Foundations.—The depôt was built upon soil of very unstable character, being situated on the site of the marshes which formerly existed beyond the eastern boulevards of Paris.

The footings consist of two parallel armoured brick walls 30 centimetres deep by 30 centimetres high by 11 centimetres thick, spaced 8 centimetres apart, as shown at the bottom of Fig. 110, the two walls being joined at the top by a slab of reinforced Portland cement mortar 5 centimetres thick. The caissons so formed support the outer walls of the building.

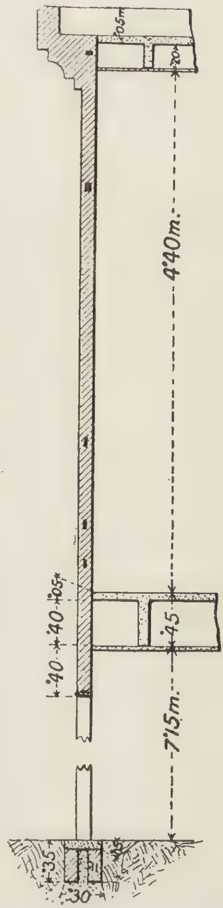


FIG. 110.—Section DD (Fig. 111).

101. Walls.—The walls of the building are only 11 centimetres thick, being of brick reinforced with a network of steel rods, the vertical bars of the reinforcement being securely fixed in the top slab of the footing. No piers or buttresses were added for the lateral support of the walls, which carry, at the height of 7.15 metres above ground level, a floor load of 3,500 kilogrammes per square metre on spans of 8.65 metres and 7.60 metres wide, in addition to the loads due to the weight of the upper portion of the structure. The walls are continued to the height of 4.40 metres above the first floor (see Fig. 108), and at the top they receive the stiffening ribs of the flat roof, the reinforcement of which is connected with that of the walls.

102. Details of Wall Construction.—Above the open side of the workshop (see Fig. 111) the reinforced brick wall descends to 40 centimetres below the stiffening ribs of the floor, and its total depth above the opening on the ground floor therefore is—

From top of roof slab to ceiling of upper storey	0.25	metres.
From ceiling to floor of upper storey	4.40	"
From top of floor to ceiling below	0.45	"
From ceiling of upper storey to top of opening	0.40	"
Total . . .	5.50	"

Below the brickwork of this wall a bar of steel plate, 40 millimetres wide by 18 millimetres thick, is fixed as an additional support, the ends of the bar being securely connected with the vertical reinforcement of the return walls and with the triangulated bracing in the thickness of the wall. This wall virtually forms a girder 5.50 metres high, which spans the opening of 15.21 metres and entirely avoids the use of the massive steel beam which would otherwise have been necessary for supporting the heavy loading previously mentioned.

Instead of reinforcing the brickwork with round steel bars forming a rectangular network in accordance with the method generally followed in the Cottancin system, flat

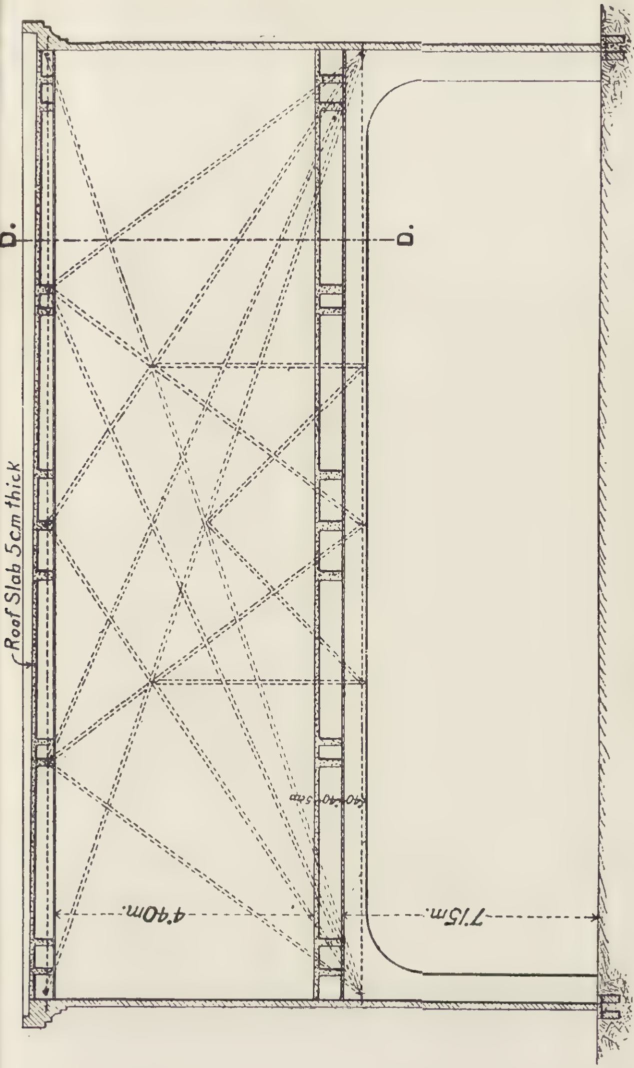


FIG. 111.—Section on CC (Fig. 107).

bars of steel plate have been used, with the dimensions of 40 millimetres wide by 16 millimetres thick, and arranged to form a triangulated system of bracing, as shown in Fig. 111. The bars are embedded in the brickwork, and their ends are securely connected with the horizontal and vertical reinforcement of the side walls and roof, and with the horizontal plate at the base of the wall over the opening.

At the points where the bars cross they are firmly bound together by steel wire ties, and where the bars are made up of more than one length the extremities are connected, as illustrated by Figs. 112 and 113, by being bent over to form hooks, which, when linked together, are held in position by a spiral binding of steel wire. When these joints are embedded in cement mortar the connection provided is as good as a weld for resisting either tension or compression.

FIG. 112.

FIG. 113.

It will be seen that the triangulated system of reinforcement, being connected with the steel bars in the adjoining parts of the building and stiffened laterally by the brickwork in which it is enclosed, virtually constitutes a truss 5.50 metres deep by 11 centimetres thick, and is capable of supporting a very heavy load.

The entrance to the coal store is spanned by a reinforced bressummer wall of similar construction, also carried 40 centimetres below the level of the upper floor.

103. Floor Construction.—The upper floor, which is of concrete-steel, consists of a series of stiffening ribs, each 40 centimetres high and 7 centimetres thick, reinforced with steel bars, the dimensions of which in the strongest members are 40 millimetres wide by 18 millimetres thick, and following the system of triangulation shown in Fig. 114. Upon and bonded with these stiffening ribs is a floor slab 5 centimetres thick, formed of Portland cement concrete

reinforced with steel rods interlaced to form a network, the meshes of which vary in dimensions according to the resistance required.

The weight of steel per square metre of the floor slab is between 10 and 11 kilogrammes. If the meshes were of uniform dimensions, and proportioned to the most severe stresses experienced, they would probably weigh about 16 kilogrammes per square metre, representing increased expenditure without any corresponding advantage.

The reinforcement of the floor ribs and floor slab is securely tied to that of the walls.

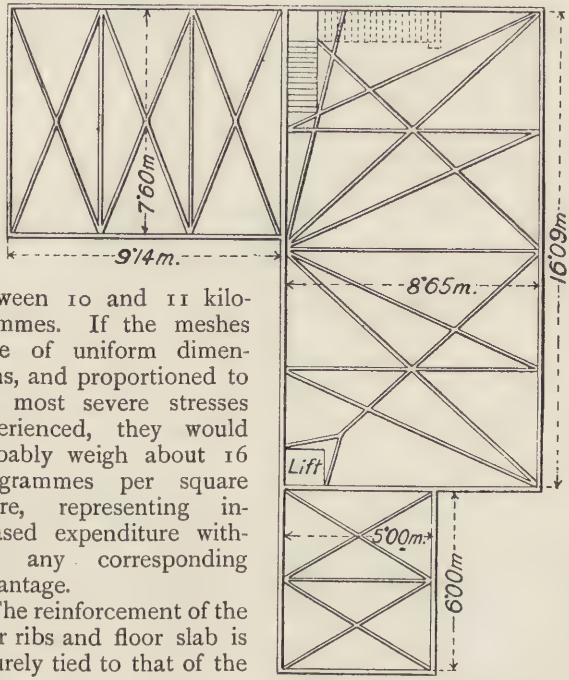


FIG. 114.—Plan of First Floor, viewed from below.

104. Roof Construction.—The ribs of the roof are 20 centimetres deep, and are arranged similarly to the stiffening ribs of the floor. Over and bonded with them is a slab of reinforced cement concrete 5 centimetres thick, all the reinforcement being connected as in the floor. Below the stiffening ribs of the roof is a ceiling of armoured plaster, which with the concrete-steel roof slab forms an

air space about 20 centimetres high over the accumulator rooms. The impermeable roof covering and the non-conducting cushion of air enclosed below it afford efficient protection against rain and variations of temperature. Figs. 108 to 111 make clear the chief features of the roof construction.

105. Floor Tests.—On the completion of the building, the floors were tested by M. Monmerque, with the full load of 3,500 kilogrammes per square metre over the two floor surfaces of 16.09 metres by 8.65 metres and 9.14 metres by 7.60 metres respectively.

The measuring instruments used were of the Manhès type, largely employed in France in connection with tests of steel bridges, and the trials gave very satisfactory results in each case.

During the same tests it was demonstrated that the walls of the building suffered no appreciable lateral deformation under the heavy loads supported and the severe strains caused by deflection of the floors. Remembering that the walls are only 11 centimetres thick, and are built entirely without stiffening piers or buttresses, this result is certainly remarkable.

The author is informed by M. Cottancin that M. Monmerque was somewhat surprised by the records obtained, not so much because the floors successfully withstood the heavy loading, as for the reason that their behaviour with regard to flexure was not in accordance with his preconceived views. M. Cottancin also states that M. Monmerque was not prepared at first to accept the theory that either floor would behave as a single structure, and believed that the panels would act more or less as separate beams or slabs. Fig. 115 is a diagram representing the three panels of the 9.14-metre by 7.60-metre floor shown in Fig. 114, and, concerning this, the suggestion of M. Monmerque was that the rectangles ABEH, HEKI, and IKML would act independently. In that case the diagram of bending moments for the span AB would be approximately as shown in Fig. 116.

M. Cottancin, however, contended that this theory was wrong, and that the rectangle ABML must be considered

in its entirety, arguing that the four surfaces HBEI, HKMI, AEKH, and IEKL in Fig. 117 would work together.

106. Discussion of Floor Tests.—The manner in which the floors behaved will be more readily appreciated by reference to Fig. 118, where PR is the horizontal, below

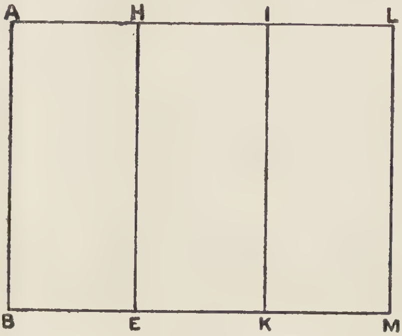


FIG. 115.

which, and parallel with it, is a line at the distance *t*, and above it two other parallel lines, one at the distance *r* and the other at the distance *s*, the latter being situated at twice the height of *r* above the horizontal line PR.

At the full load of 3,500 kilogrammes per square metre,

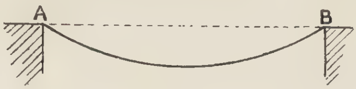


FIG. 116.

the point *a*, at the left-hand support, rested upon the line PR (refer to Fig. 117 for the position of the point *a* on the floor surface). The point *b* rose above the horizontal to the distance *r*; the point *c* descended below the horizontal to the distance *t*; the point *d* rose above the horizontal to the distance *s* $(= 2r)$; the point *e* descended below the horizontal to *t*; the point *h* rose to *r*; and the

point *i*, at the right-hand support, remained upon the line PR.

Hence, instead of being deflected under the load, *b* and *h* were raised, and *d* was raised to a height double that of *b* and *h* above the horizontal. This result seems quite

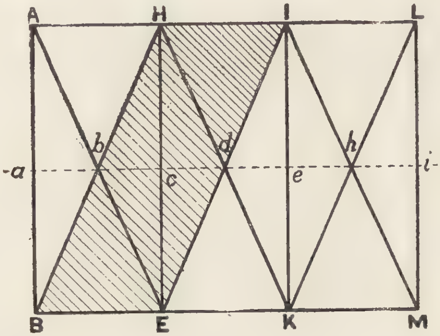


FIG. 117.

reasonable, for *c* and *e* in the members HE and IK (Fig. 117) having dropped, it follows, in accordance with the laws of equilibrium, that *b* and *d* and *h* and *d* must rise. Further, as *d* was raised by an effort double the value of that acting upon *b* and *h* respectively, *d* was raised to twice the height to which *h* and *d* were raised.

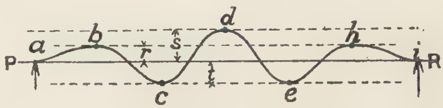


FIG. 118.

107. Theory of Cottancin Floors.—This article is taken from a demonstration sent to the author by M. Cottancin, who explains therein the theoretical basis of his system of floor design. M. Cottancin points out that action analagous to that described above is observable in connection with the construction of lock and dock gates,

which, whether made of timber or of steel, consist of plane vertical surfaces, stiffened by bracing, consisting of projecting ribs, and proportioned in accordance with the rule that the maximum pressure occurs at one-third of the height, such gates being made in pairs meeting at the middle of the opening to be closed.

It has been found that when the bracing possesses a certain resistance this resistance causes variation of the bending moments between the point of maximum thrust and the top of the gate, although the maximun thrust is always exercised at the same point.

This view is illustrated by the case of a bridge girder, where the maximum bending moment is not necessarily to be found at the point of maximum load.

It follows that the resistance of the horizontal stiffening ribs can be ascertained with reference to the vertical plates or planks of a gate, so that the maximum effort at any point whatever shall always produce a maximum bending moment at the upper part of the gate and on the centre line between the vertical walls of the gateway.

It is evident that under a static load the character of the bending moment is not changed and the most favourable conditions of resistance are thereby ensured, because the various parts of the structure are not alternately in tension and compression, as in the case of a bridge subject to rolling loads.

If the maximum bending moment of a dock gate occurred normally at one-third of the height it would be displaced with the increase of pressure, giving rise to conditions similar to those graphically represented in Fig. 119. Upon the section *mn* in this diagram, where the maximum bending moment is at AB, there is compression on I and tension in II and III, whereas in the case represented by Fig. 120, and the points of the curves I, II, and III in the section *mn* are always in tension.

In the Cottancin floor system the stiffening ribs—corresponding with the bracing of a dock gate—are proportioned to the resistance with reference to the floor slab—corresponding to the planking or steel plates of the dock gate—so that the bending moment is always maximum at

a point such as *b* at the centre of *ac*, in the surface MN*cba* (Fig. 121). But we may take a surface M′N′*cba* intimately connected with the surface MN*cba*, and arranged so that the surface M′N′*cba* shall be able to transmit the maximum

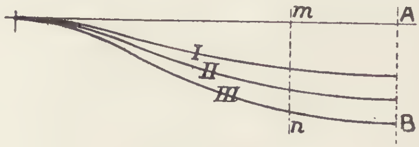


Fig. 119.

bending moment at *b*. Then a force P, acting in a downward direction at the centre of the surface MN*cba*, will not

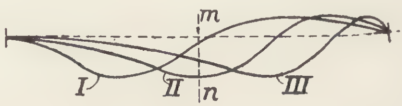


Fig. 120.

be able to depress the line *ac*, as shown by the line *abc*, and the line *ac* will follow the modified direction *ab′c*, because

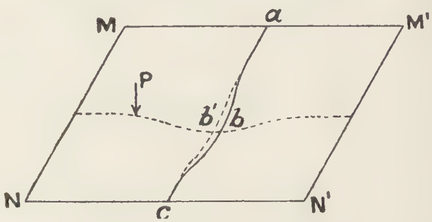


Fig. 121.

the surface M′N′*cba* offers resistance to the sinking of the point *b* in the surface MN*cba*.

The same line of reasoning is extended to ribs such as MN′, M′N in Fig. 122, where the surface is assumed to be formed so that the downward force perpendicular to a line

as *rs* (Fig. 123), and acting at any point whatever in that line, shall still leave the maximum bending moment at the centre. Thus we have *rts* for the curve of bending moments with the force at P, $rt's$ with the force at P′, $rt''s$ with the force at P″, and $rt'''s$ with the force at P‴. Further, in a floor panel braced by means of stiffening ribs passing

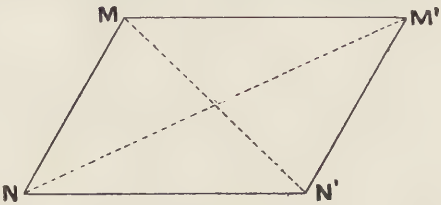


FIG. 122.

from one corner to another (as shown in Fig. 122, any downward force P acting upon any point whatever of the surface MM′N′N will always cause the maximum bending moment due to that force to occur at the point of intersection of the ribs MN′ and M′N.

Hence M. Cottancin points out that the critical points

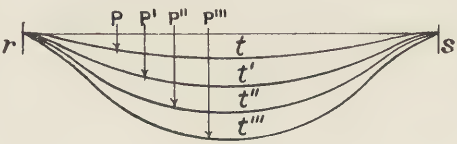


FIG. 123.

in a floor such as that illustrated in Fig. 117 are *b*, *d*, and *h*, where the greatest resistance to bending moment is furnished by the diagonal ribs, and argues that the depression of parts not so stiffened causes the elevation of the stiffened parts.

This view is supported by the tests conducted at the depôt in the Rue de Lagny.

Tests conducted at L'Ecole des Ponts et Chaussées show

further that floors constructed in the manner here described behave as if their outer edges were securely fixed, while at the same time they are merely supported in the ordinary way.

108. Cottancin Reinforcing Network.—The steel rods employed in the Cottancin system for reinforcing brick walls and concrete slabs are applied so as to form a reticulated network, such as that represented in Fig. 124.

The rods are passed through holes in the bricks, or embedded in concrete. Drawings illustrating the arrangement of the network will be found in Figs. 129 and 130.

This article relates only to the principles relied upon in floor construction.

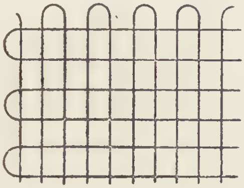


FIG. 124.

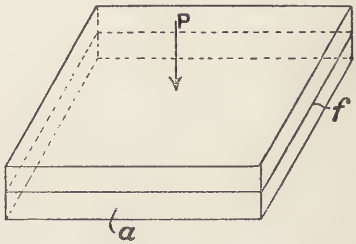


FIG. 125.

In the first place, it is recognised that one effect of vertical force, applied at any part of the surface, is to cause lateral expansion, resulting in pressure of the mortar or concrete against the strands of steel constituting the reinforcing network. Fig. 125 represents a prism of mortar or concrete in a floor slab reinforced by a network of steel bars, P the applied force, and *f* the reinforcement surrounding the prism of which *a* is one side. The effect of binding the prism is similar to but less in degree than that due to the hooping of concrete in column construction as recommended by M. Considerè.

In addition to the restraining influence of the steel rods the prism *a* is further reinforced by being imprisoned, as shown in Fig. 126 in the middle of a series of eight similar

prisms, lettered *b*, each of these being surrounded by a mesh of the network, and the whole series by the hooping *f′*, whose resistance against lateral bulging is increased by connection with the strands of the network passing between the prisms *a* and *b* in directions both parallel and perpendicular to every side of the group. Thus the prism *a* experiences from the eight prisms *b* a reaction equal to the action producing lateral expansion, less a diminution proportionate to the elastic deformation of the prism. If the prism were not reinforced laterally in the manner described it might fail under the force P, whereas, being treated in the manner shown, action is neutralised by reaction, with the result that the prism has merely to

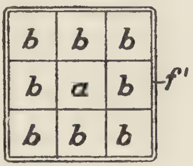


FIG. 126.

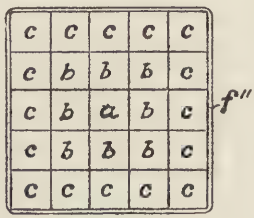


FIG. 127.

withstand stress equal to the difference between the action and the reaction. Assuming this difference to be equal in value to the elastic deformation of the material, it follows that a great accession of strength must result.

Further, it must be remembered that this small proportion of the original force P is distributed among the surrounding eight prisms *b*, and in turn the group of nine prisms represented in Fig. 126, when surrounded by other prisms *c*, as in Fig. 127, only transmit to each of the sixteen prisms a still smaller proportion of the elastic deformation of the prism *a*. It will be observed that the twenty-five prisms in Fig. 127 are hooped by the band *f″*, which is tied laterally by the network, as previously explained. Of course, only a single downward force P is here considered, while in the case of a floor under uniformly distributed

load there would be the equivalent of a force P on every prism into which the floor is subdivided by the meshes of the reinforcement, but this multiplication of the force does not in any way effect the beneficial influence of the system of reinforcement described.

MAISON DE RAPPORT, PARIS

109. General Description.—An excellent example of reinforced construction as applied to dwelling-houses is to be found at No. 29 Avenue Rapp, Paris, a street running from the Avenue de la Bourdonnais, on the north-eastern side of the Champ de Mars, to the Pont de L'Alma.

Fig. 128 is a plan illustrating the general arrangement of the building, which was designed by MM. Combes et Lavirotte, a Parisian firm of architects, in accordance with the Cottancin system of reinforced construction. In order not to depart altogether from customary methods of construction, the architects decided to employ stone for building the exterior walls up to the first storey on the main façade and to ground level elsewhere, except in the case of some walls which are constructed in reinforced brick down to foundation level.

The upper portion of the building is constructed in reinforced brick and reinforced concrete.

110. Foundations.—Those walls which are of reinforced brick, as well as four interior columns, are built upon caisson foundations, somewhat similar to those described in Articles 91 and 100, the sides of the caissons being of reinforced brick 11 centimetres thick, with a cover formed by a slab of reinforced concrete 5 centimetres thick; the footings so formed being generally 30 centimetres wide by 40 centimetres deep.

Fig. 129 is a perspective by which it will be seen that the sides of the caisson consist of five courses of brick, each 7 centimetres high by 11 centimetres wide.

For the four interior columns the foundations consist of caissons 70 centimetres square by 40 centimetres deep, the upper slab being of reinforced concrete 5 centimetres thick. Fig. 130 is a perspective sketch of a typical foundation, with

a portion of the column. The sides of the caisson consist of four brick walls 11 centimetres thick, all reinforced by

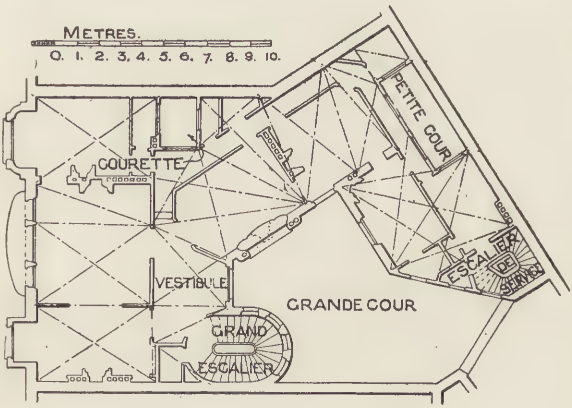


FIG. 128.—Maison de Rapport, Paris (Plan).

steel wires of 4.4 millimetres diameter, these being securely connected with the steel network of the cement slab.

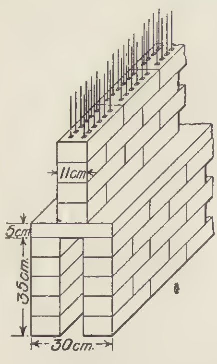


FIG. 129.

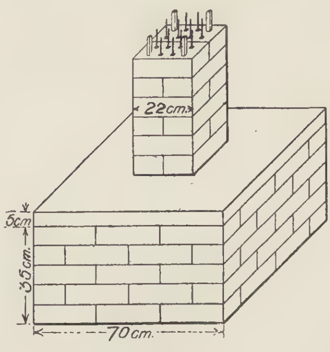


FIG. 130.

111. Walls.—The bricks in the wall standing upon the caisson foundation illustrated in Fig. 129 are 11 centimetres

wide, and are reinforced by steel wires of 4.4 millimetres diameter (No. 20 French gauge), passed through holes which are afterwards filled up with cement mortar. These wires are tied with horizontal wires of the same gauge laid in the joints to form a network capable of taking all tensile stresses developed in the structure.

The number of wires used in the vertical holes and in the joints of the brickwork is proportioned so that adequate resistance shall be ensured in all parts of the wall. In places where the number of wires necessary may be too great to permit them to be woven into a network the practice is to use a bar of steel of the required dimensions in place of a bundle of wires.

The reinforced brick walls of the two small courts, the grand staircase, the service staircase, and the lift enclosure, all built in the manner here described, extend from the basement to the eaves—that is, through eight storeys in all.

The interior walls of the house are of two kinds—one built hollow and the other consisting of a simple partition of brick 11 centimetres thick.

The outer walls of the building consist of an inner and outer skin of reinforced brick with an interior air space. Upon the courtyard—*grande cour*, Fig. 128—the outer wall partition is built of white brick, relieved at some parts with red brick, and at others with enamelled brick, either plain or moulded, as required to suit the general scheme of the façade. The interior partition is in ordinary brick, which is employed also for the headers which bond the inner and outer skin of the construction.

In the principal façade upon the Avenue Rapp the outer skin of the hollow wall is built of moulded stoneware blocks reinforced in the same manner as that employed for ordinary bricks. Fig. 131 is an outline section through the principal façade, showing the wall construction.

All the inner surfaces of the outside walls and the partition walls are finished suitably to the purposes for which the various apartments are intended.

The kitchen, sculleries, and other domestic offices, as also the lavatories throughout the house, are finished with

wall surfaces in white enamelled brick. No wall tiles or other materials for forming a veneered wall surface have been used, the architects rightly believing these to be undesirable because of their liability to become detached in course of time. The pantries have walls of reinforced brick and cement.

112. Hollow Block Walls.—In some parts of the building, where it was desired to provide adequate resistance against the conduction of heat without resorting to double walls, the walls are built of hollow earthenware blocks of the section illustrated in Fig. 132. The face of the block is at the top in this drawing, and the space inside is filled with non-conducting material, the interior wall surface being suitably finished. The blocks are moulded with grooves for the reinforcing network.

113. Columns.—Four columns in the vestibule, standing upon caisson foundations, as illustrated in Fig. 130, at the level of the basement floor, are of reinforced brick with a cross section of 22 centimetres square up to the level of the ground floor. The columns are built with two bricks in each course, the joints of alternate courses being at right angles so as to give a proper bond. The reinforcement at the corners consists of steel bars, and in the other vertical holes of 4.4 millimetre steel wire. The vertical wires are interwoven with horizontal wires laid in the joints and the four corner bars are connected by diagonal ties.

In the vestibule the columns have been brought to cylindrical form by an outer coating of cement, stuccoed

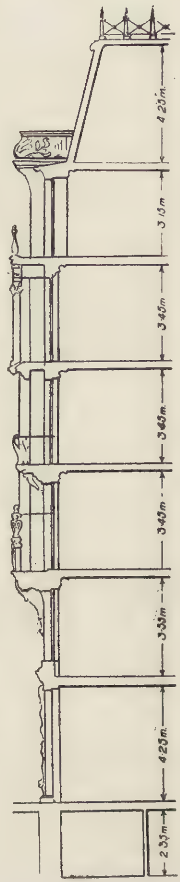


FIG. 131.—Section of Main Façade.

and decorated to imitate the marble panelling of this apartment.

114. Floors.—All the floors of the building, from the ground floor to the top storey, are formed of concrete-steel, the same material being used for the flat portions of the roof and for the paving of the courtyard, which is intended for use by carriages and motor-cars. Each floor is finished with a slab of cement-steel 5 centimetres thick, in which the network of steel is disposed in meshes of dimensions graduated in proportion with the resistance required at each point.

Fig. 132.

Each panel of reinforcement is connected with those adjoining by hooking and binding together the separate wires and bars of the metal, so that the entire surface of each floor really consists of a single monolithic slab of cement, the reinforcement of which is one connected system, formed of several smaller parcels woven together.

The floor panels are laid upon concrete-steel stiffening ribs 20 centimetres deep by 5 centimetres wide, arranged in accordance with a system of triangulation which varies from floor to floor, and is in every case designed to suit the precise conditions of loading.

The stiffening ribs are termed by the originator of the system *épines-contreforts*, or counterfort-spines, with the object of indicating that they are not to be regarded as independent beams or joists, but rather as members analogous to the ribs and backbone that reinforce the body of a vertebrate animal.

In the case of the building now under consideration all such members were made beforehand in the basement, with the threefold object of ensuring satisfactory execution, of obviating the necessity for the costly system of beam moulds and struts usually employed in the construction of concrete-steel floors, and of avoiding enforced stoppages of work during the hardening of concrete in the beams and joists.

The ribs were moulded flat, as sketched in Fig. 133.

Here the dimension 20 centimetres represents the depth of the rib, but, as the member is moulded on its side, the depth has no influence on the quality of the work, and the concrete can be thoroughly tamped. The reinforcement includes two bars of steel, one at A and the other at B, longitudinal wires *a* and transverse wires *b*, each of the latter being tied to the bars A and B, and forming a loop *c* projecting through the top surface of the rib. The area of steel in the bars A and B depends upon the duty of the rib. The wires *a* are in spiral coils of flattened form, and the wires *b* are straight, the diameter in each instance being 4.4 millimetres. The projecting loop *c* is for the purpose of incorporation in the floor slab, and forms part of its reinforcement.

At points where two ribs cross each other a "halving-

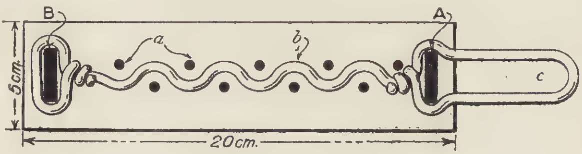


FIG. 133.

joint" is adopted somewhat similar to that used in carpentry. To make this joint, one rib (as N in Fig. 134) is first fixed in position, and below it is placed another rib P made without the bar A.

Then a notch from the top down to the bar B is cut in the concrete of the second member at the point of junction. The wires (*a*) are also cut, and P is lifted so that N penetrates as far as the bar B.

Then at the point of intersection the concrete of the rib N is cut away from A to B, and the bar B of one rib can be brought into contact with the bar B of the other as indicated by the dotted outlines at the middle of Fig. 134. Then the two bars are tied together. This is done by means of two spiral windings of steel wire, the windings crossing each other at right angles.

The wires *a* of the rib N have not to be cut, and the

ends of those in P which have been cut are jointed together, thus restoring the arrangement which existed before they were disturbed.

Cement mortar is then applied to fill up the space of the concrete removed from the two ribs at the point of intersection, and the bar A, omitted from the rib P, is passed through the loops *c* of that rib.

At the point of junction the bars A of the ribs N and P are securely wired together to make a firm connection. This joint is also covered with the mortar, through which project the loops *c*.

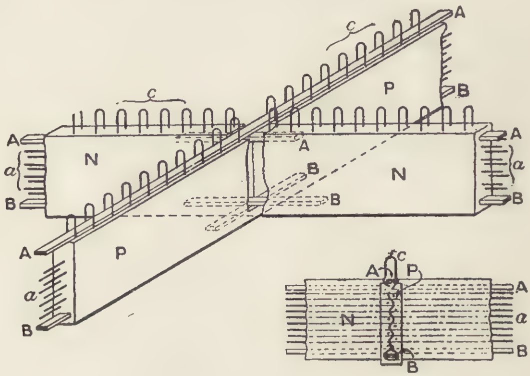


FIG. 134.

FIG. 135.

In Fig. 134 the loops at the point of intersection of the two ribs are omitted for the sake of clearness. As the bars A and B of the two ribs are not halved together it is clear that their top surfaces cannot be brought to the same level, but this slight difference is made up by the thickness of bar A of the second rib, which rests upon the top surface of the concrete.

In this manner a joint is made without impairing the strength of either member.

Fig. 135 represents N in elevation and P in section after the joint has been finally completed, and from this

sketch the relative positions of the bars A and B can be readily understood.

When the mortar of the joint is well set the spaces between all the stiffening ribs are spanned by plates of reinforced plaster 2 centimetres thick. These plates are supported by cleats of wood wedged or otherwise held in position against the sides of the ribs, and over them is spread a network of steel rods. This network is made beforehand in a suitable workshop, and its meshes pass between the loops *c* projecting from the stiffening ribs. Next, the loops are bent downwards successively from one end to the other of each rib, so that they form a kind of chain which imprisons the network spread over the plates of plaster. In this way the reinforcement for the floor slab is formed, and the armoured plaster plates constitute the centring.

The material of which the floor slab is formed consists of 1 part of Portland cement and 2 parts of sand, the first layer being mixed very wet to enable it to pass freely beneath and between the steel wires and rods; while, to assist the penetration of the concrete, the network is lifted up by means of hooks. The second layer of concrete is mixed very dry, in order that it may suck up the excess of water in the bed below. In this way a homogeneous floor slab 5 centimetres thick is formed over the whole surface to be covered.

In places where it is desired to lay wood flooring, strips of timber are bedded down upon the first layer of mortar, and are held in position by nails driven into the sides of the strips so as to project head downwards at an angle of about 45 degrees. The heads of the nails catch in the meshes of the reinforcing network, and are securely held by the surrounding concrete. Floor boards can then be nailed down in the ordinary manner.

It should be added that some of the stiffening ribs are made with beaded projections along the lower part of each side, thus forming ledges, which are used in the first place for supporting the cleats by which the plaster plates are held in place during the construction of the floor slab. After the slab has set the cleats are removed and the

sheets of armoured plaster are lowered until they rest upon the projecting ledges of the stiffening ribs, and there remain as the permanent ceiling. Thus, when the joints are plastered up, a hollow ceiling is formed, confining an air cushion, which minimises the transmission of sound and the conduction of heat.

115. Roof Construction.—The roofing of this building is constructed in a manner generally similar to that adopted for the floors and ceilings, and between the upper and lower surfaces a cushion of air is confined which has the effect of keeping the rooms immediately below moderately cool in summer and warm in winter.

With the exception of the mansard slopes (see Fig. 131), the roof surfaces of the building are flat, and, with the object of preventing the formation of cracks in the concrete-steel panels, a special method of construction was adopted, the principle of which is illustrated by Figs. 136 and 137, the former representing the effects to be avoided and the latter the effects of the method actually followed.

The top diagram of Fig. 136 shows two stiffening ribs of similar construction to that illustrated in Fig. 133. Between the ribs is a plate of armoured plaster wedged in position, for the purpose of forming centring for the deposition of the concrete-steel panel. This plate, being deflected under the weight of the material deposited thereon, assumes a curved form, as represented diagrammatically in the sketch. The result is that when the concrete layer has been finished with a level surface the slab has a section like that in the middle diagram of Fig. 136, and when used by workmen, or by the occupants of the house desiring to sit in the open air, the panels will be deflected at the middle, as in the bottom sketch, causing, or tending to cause, cracks at points where the upper part of the section is in tension. Although such cracks might not be dangerous from a structural point of view, they would certainly permit the percolation of water in rainy weather, and upon a roof so constructed it would be wise to lay a covering of zinc or lead to guard against inconvenience and damage in the rooms below.

A better system of construction, and that followed in this

building, is illustrated in Fig. 137. Here the upper diagram shows two ribs, which are similar to those in Fig. 136, but have triangular flanges at the top as well as at the bottom. Between these the plate of reinforced plaster is wedged in such a way that it presents a convex surface when viewed from above. When the concrete is deposited this arch is depressed slightly at the centre, but still retains a curved form, and when the top surface of the roof panel has been levelled off the section resembles that in the middle drawing

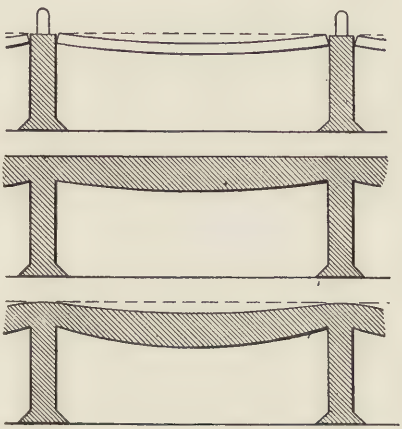


FIG. 136.

of Fig. 137. The imposition of load upon the centre of one arch naturally has the effect of causing depression at the crown, but cannot result in rupture at the abutments, and the elevation of adjoining arches of the structure by expansion, as suggested in the bottom sketch of Fig. 138, is equally harmless.

As shown in Fig. 131, the roof is of the mansard type, the slopes being built of reinforced brick and, consisting of two walls with an intervening air space, intended, as in the case of the flat surfaces, to modify the influence of exterior climatic conditions. Upon the outer brick wall of the roof

slope that faces the Avenue Rapp enamelled tiles are fixed by means of Portland cement mortar.

The rain-water gutters at the foot of the mansard slopes are formed in reinforced cement mortar moulded in forms suitable to the architectural features of the building and tinted in black ochre, thus giving an appearance resembling that of slate or cast iron. Gutters are also formed in the flat portions of the roof system.

The dormers which occur in some parts of the mansard

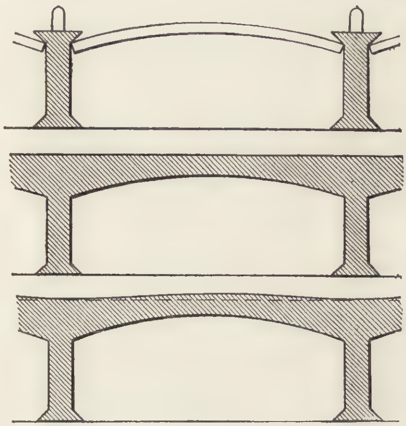


FIG. 137.

roof are built of reinforced brick, and the reveals are of the same construction. Those overlooking the main courtyard are finished by a rendering of cement coloured to the required tint, while those on the main façade are covered with ornamental stoneware reinforced by steel wire. All skylight openings in the roof are constructed in reinforced brick.

116. Cantilever Construction.—We now direct attention to a most interesting constructive feature, consisting of a portion of the building which overhangs the main courtyard, as indicated by the shaded area in Fig. 138. This

projecting angle constitutes a great corbel six storeys in height, and extending from the first floor to the top of the house. The column, which is shown in elevation in Fig. 139, is of stone, and appears to support a very great load. But in reality it has no work to do, and might have been omitted entirely without impairing the stability of the structure. It was included chiefly for the sake of appearances, and as a matter of fact the column was not put in its place until the whole of the work above had been completely built.

Fig. 139 is intended to make clear the principle involved in this bold piece of structural engineering. The lower portion of the corbel is represented by the lines AB, BC, these being the lower boundaries of the brickwork which

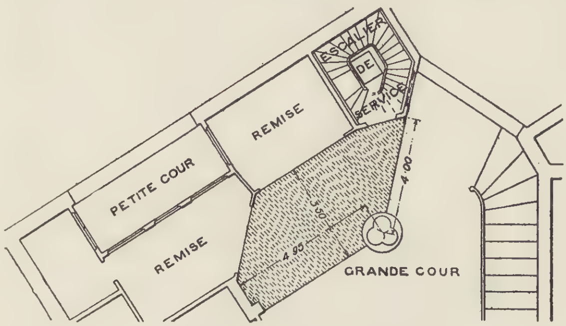


Fig. 138.—Part Ground Plan.

actually transmits the loads to the walls of the main building. It will be understood that, as the two outer walls of the corbel are of reinforced brick, they have been thoroughly connected by means of steel network with the walls of the main building, to which they are further tied by the floors of storeys Nos. 2 to 7. These floors, permeated by steel network connected with the network of the walls on every side, are in effect horizontal, very wide, and shallow tie-beams. Further, we have the cross walls of the various rooms, and these also are tie-beams, very deep and thin. Taking into account the vertical outer walls, the

inner cross walls, the horizontal floors, and the roof construction, this projecting angle of the house is seen to be nothing more than a huge cantilever braced in the most efficient manner, and quite independent of any support beyond that derived from the building to which it is secured, and with which it is incorporated.

We will turn next to the details drawn in broken lines. In the first place, we have two triangular wall surfaces which, taken together, form the area ABC in Fig. 139.

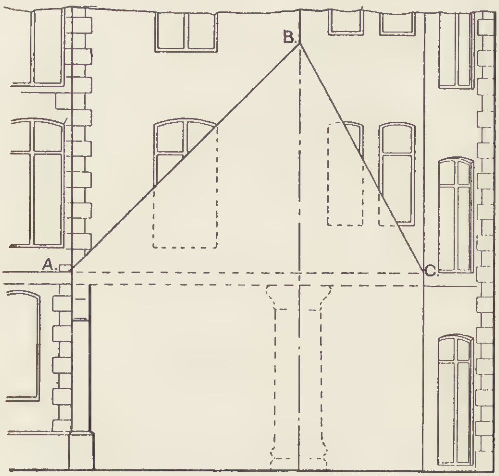


FIG. 139.—Cantilever Construction.

These portions of the wall are in reality suspended from the upper part of the reinforced brickwork, and are thoroughly bonded with it in the ordinary way, as well as by the network of steel wires passing continuously through and between the bricks. The floor of the first storey is also suspended at the outer edges, while along the two inner sides it is supported by the walls of the building as the other floors are supported. In this drawing the suspended portions and the column are merely drawn in dotted lines for the purpose of identification.

When the great console, with its dependent walls and floor, had been built the floor of the first storey, with the whole of the six storeys above, remained suspended in mid-air until the stone column was erected in its place to give this portion of the building a reasonable appearance of security, and to ensure outward compliance with the form of construction to which the eye is accustomed.

Although the designer entirely disregarded the support afforded by the column, it must not be forgotten that this member is really capable of taking the weight of the projecting angle and, being placed beneath it, must take its share of the load, thereby relieving the reinforced brick-work of strain and adding very materially to the strength of the construction as a whole.

117. **Braced Gallery Girder.**—Another singular piece of design at the front of the building is to be found in an arch, of 7 metres span, over a large window of the third storey. This arch appears to support a gallery above, but in reality does nothing of the kind, being supported by the gallery, which is about 4 metres high and, stretching from side to side of the façade, is practically a braced girder of great strength.

118. **Staircases.**—Both the principal and the service staircases are formed with an inclined plane of reinforced cement mortar 5 centimetres thick. Above the surface of this slope are two stiffening ribs, one placed at each side and corresponding with the customary "string boards." The treads, also of reinforced cement mortar, were moulded so as to give a channel section, and, of course, are set in position with the flat side uppermost. Above the principal staircase is a water tank for the operation of the passenger lift, the tank being constructed of reinforced brick and cement.

119. **Pavement Lights.**—For lighting the basement, pavement lights are provided in the three courts. Pavement lights of the kind ordinarily used in Paris having frames of T-bars, as in Fig. 140, naturally block an unnecessarily large proportion of the light available. To obviate this disadvantageous feature the system illustrated in Fig. 141 was introduced by M. Cottancin. The frame-

work consists merely of steel wires 4.4 millimetres diameter, which pass along grooves in the sides of the glass panels and are woven together so as to constitute a network capable of supporting all the weight likely to come upon the construction. As the wires are of small diameter they obstruct very little light, and, being laid in the circular holes formed by two semi-circular contiguous grooves, the glass panels can be brought close together and jointed with cement.

Fig. 140.

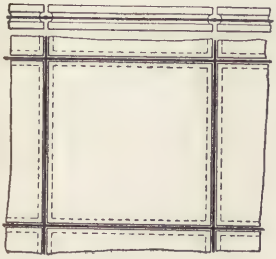


Fig. 141.

120. Chimneys and Ducts.—A distinctly novel arrangement in this building is that some of the chimney flues, as well as all the ventilation ducts, pass entirely within the space afforded by the hollow-wall construction. Between the inner and outer walls two series of metal lathing are fixed at the required distances apart, and the lathing is plastered over to form the conduit. In this way all flues required for the rooms in the front of the house are carried up inside the walls of the principal façade, while others are situated in the thickness of the inner walls. The walls containing smoke flues also support the floors of the building, an arrangement that would clearly be inadmissible in ordinary construction. The flue of the hot-air stove is of similar construction, and close to it are situated the water pipes and drain pipes proceeding to and from various parts of the house. The drain pipes are of Doulton ware placed in plastered casings formed between the inner and outer walls of the building, and embedded in cement.

121. Water Tanks.—The kitchen water tanks are built of enamelled stoneware tiles moulded with grooves, like those of the glass panels in the basement lights, and are similarly connected with wire reinforcement and jointed with cement. The interior face of each tank is formed by

the enamelling of the tiles, and the outside is rendered in cement or plaster.

L'EGLISE DE SAINT JEAN DE MONTMARTRE, PARIS

122. General Description.—The church of St. Jean de Montmartre is a remarkable example of reinforced construction, and, so far as concerns boldness of design, it probably has never been surpassed.

Owing to the fact that Montmartre is a thickly populated and poor quarter of Paris it was necessary to build a large church at the lowest possible cost. Accordingly, M. A. de Baudot, architect to the archiepiscopal district of Paris, obtained three estimates for consideration, relating respectively to masonry, steel-framed, and reinforced brick construction, the approximate cost of the three projects being—

(1) Stone masonry	£155,600
(2) Steel - framed construction cased in masonry, on the American system .	£45,000
(3) Reinforced brick and cement construction, on the Cottancin system .	£14,000

After these estimates had been considered by the diocesan authorities, M. de Baudot was instructed to accept the last-mentioned.

Figs. 142, 143, and 144 will serve to make clear the main features of this building. By reference to Fig. 144 it will be seen that the church is of irregular plan, the total length being 51 metres and the width across the nave and aisles 20 metres, not taking into account the small chapels and other projecting parts. The height of the outer walls is 35 metres, and the thickness of the reinforced brickwork of which they are built is only 11 centimetres ($4\frac{3}{8}$ in.). Support for the entire structure is afforded by cylinder foundations, on which are built reinforced brick columns with a central core of concrete.

The building comprises two storeys, one being the crypt, 10 metres high, and the other the church proper, 25 metres high to the top of the walls (see Fig. 143). The floor of the crypt is level with the tops of the cylinder foundations, the stiffening ribs and concrete slab of this

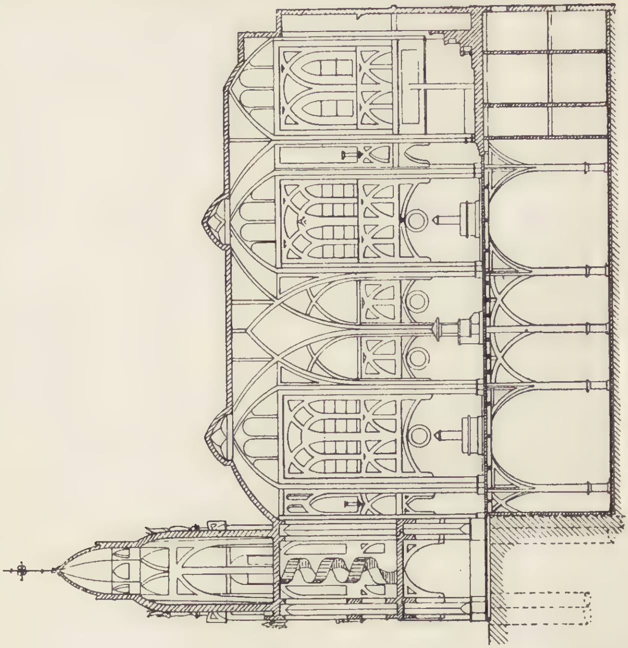


Fig. 143.

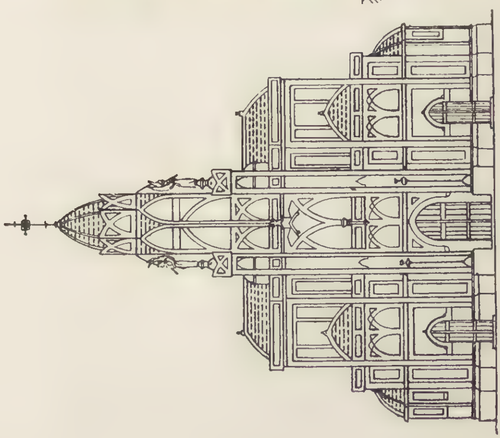


Fig. 142.

L'Eglise De Saint Jean de Montmartre, Paris (Front Elevation and Longitudinal Section).

floor connecting the whole of the foundations so that they form a complete structure.

The reinforced brick columns are carried up for the support of the church roof, and below the level of the upper floor they are provided with arched ribs, which form a vaulted roof to the crypt. In the church there are galleries and a balcony, which are supported by ribs springing from the columns.

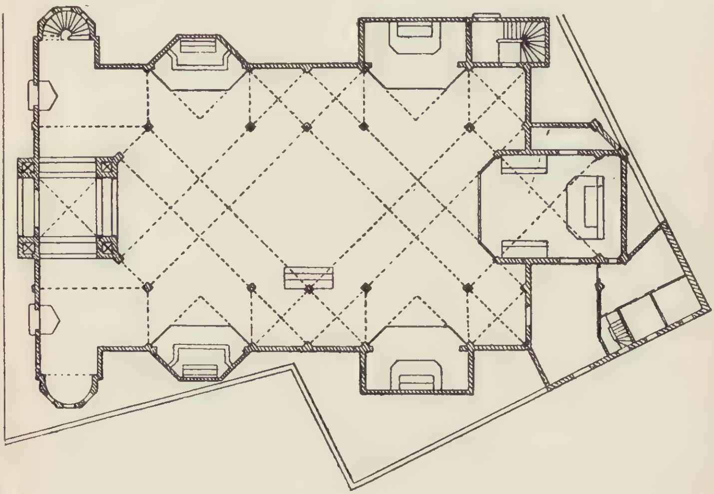


FIG. 144.—Ground Plan.

The roof of the church is a system of flat domes formed of curved ribs of reinforced brick, with connecting layers of concrete 5 centimetres thick, the whole being reinforced with a reticulated system of steel bars.

Although the walls receive support in the first instance from the connected system of cylinder foundations, they are so tied together in every part, and so connected with the columns, floors, and roof, that they are self-supporting to a very great extent, and are able to span considerable openings without intermediate piers or girders.

These are the main features of the structure, which, owing to the exceedingly daring character of its design, has somewhat undeservedly been designated as "la folie du siècle."

123. Foundations.—Considerable difficulty was presented by the peculiar conditions of the site. Fig. 145 is a block plan by which it will be seen that the church is between two buildings 20.62 metres apart, and, as the exterior width

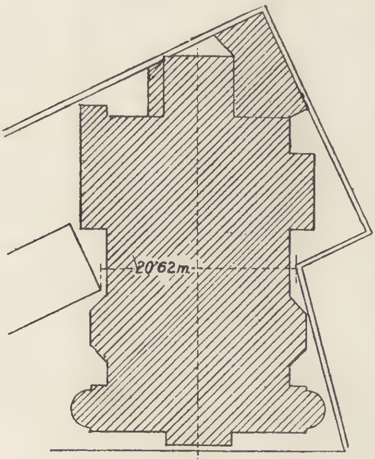


FIG. 145.—Block Plan.

of the church at this part is 20.22 metres, each of these buildings approaches within 20 centimetres of the church.

Thus it was impossible to provide for extension of the footings or to strengthen the walls by means of external piers or buttresses, the space available being, in fact, barely sufficient for the width of the building itself.

Notwithstanding the inadmissibility of counterforts, it would have been quite practicable, if the nature of the soil had permitted, to form the foundations by supporting the structure upon a general platform consisting of a series of shallow steel-cored caissons, connected by reinforced brick ribs and a concrete-steel slab.

That method is quite suitable for soils of low bearing power, but it is necessary that the ground should be uniform in character, even if of unstable quality.

In the case of the site at Montmarte the subsoil consisted of nothing better than loosely compacted quarry débris, exceedingly liable to form cavities of considerable size if any movement of the material took place. The difficult nature of the problem was further accentuated by the unfavourable profile of the site, of which a rough longitudinal section is given in Fig. 146. At the point C the level for the crypt floor was at 1.60 metres above the passage of l'Elysée des Beaux-Arts, which forms one boundary of the site. Then came a piece of comparatively level ground CA, after it a slope AB, consisting of quarry débris, and next, at 12.00 metres above the passage of

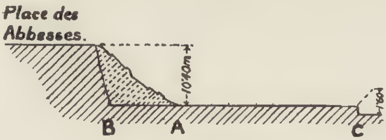


FIG. 146.—Profile of Site.

l'Elysée des Beaux-Arts, the Place des Abbesses, on which the front portion of the church now stands.

The ground plot as a whole rests upon a stratum of argillaceous gypsum, and the part between the points A and C was found to be traversed by deep fissures, descending in some cases almost to the level of the River Seine, and filled with rubbish.

In consequence of these unpromising conditions it was decided to construct a platform with cylinder foundations of depth suited to the varying quality of the substratum at different parts of the site.

As stated above, the structure is carried on reinforced brick columns, the positions of which are indicated on the ground plan (Fig. 144). Beneath each of these points a well was sunk for the reception of a hollow cylinder of reinforced concrete. After completion, the cylinders were filled to the height of 8 metres more or less with earth ex-

cavated during the sinking of the wells, and upon the filling of each a slab of reinforced concrete, 5 centimetres thick, was formed to close the cylinder at 4 metres below the surface and to form a column base-plate, the reinforcing bars of the slab being firmly connected with those of the concrete steel cylinder.

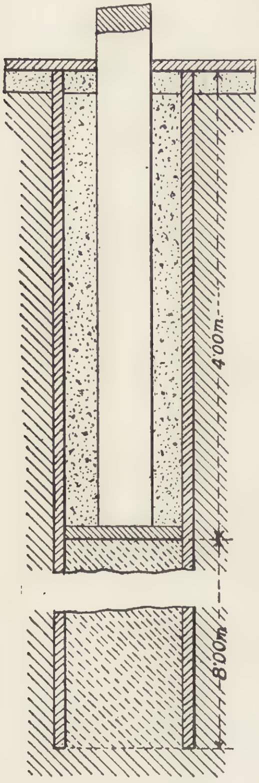


FIG. 147.—Cylinder Foundation.

Upon the reinforced panels the columns were built up, and then the upper part of each cylinder was filled with ordinary concrete. Fig. 147 is a section representing a complete cylinder foundation with its filling of rammed earth, part of a column resting upon the base-plate and held firmly in position by a filling of concrete, and a small portion of the floor system of the crypt.

By adopting this system of foundation, although at the depth of 12 metres the ground was still composed of quarry refuse, the bottom of the cylinders was lower than the galleries of the disused quarries, and there was no longer any risk that dangerous cavities would be formed by shifting of the underground material.

At the distance of 30 centimetres below the top of the cylinders a triangulated system of reinforced brick ribs was built from cylinder to cylinder, and the reinforcing bars of these were connected with a network of steel extending over all the ribs, this network being embedded in a layer of concrete forming a continuous floor slab for the crypt.

The ribs and slab connect all the cylinders, and thus a

monolithic foundation was secured, representing a combination of the cylinder and the caisson systems.

Further strength was obtained by securely tying together the reinforcement of the cylinders, the column base-plates, the columns, the ribs, and the floor slab, all these connections being made during the execution of the concrete and brick work.

The shallow parts of the foundation are quite adequate for supporting the light loads coming upon them, while the cylinders, forming cells of great depth, are able to withstand the heavy loads due to the main framework of the church.

124. Columns.—The columns, commencing at 4 metres below the top of the foundation, rise first to the under side of the floor which covers the crypt, the sides of the column being parallel and perpendicular to the axis of the building.

At the height of about 5.50 metres above the crypt floor curved ribs spring from the sides of the columns and pass from one column to another, as shown in Fig. 143. These ribs form the vaulted roof of the crypt.

As the ribs are merely extensions of the columns, and as one neutralises the thrust of another, there is practically no lateral force calculated to cause flexure of the columns. Each column acts the same part as the pillar carrying the beam of a pair of scales, as the tower of a suspension bridge, or as one of the steel towers of the Forth Bridge, supporting two cantilevers of equal length and weight. The result is that the column remains in compression, and the forces tending to produce flexure are negligible.

In the church proper the columns take a turn of 45 degrees—that is to say, each column is twisted at the floor level. Full details of this feature are given in Article 125.

The principle of branching out the columns is applied in the church on a far larger scale than in the crypt, and reference to Fig. 143 will show that the columns and their ribs, extending like the branches of a tree, carry the greater part of the roof structure. In fact, their spreading arms are the ribs forming the framework of the domed roofing system.

Thus it will be realised that the main idea of the construction is that the bulk of the weight is carried by stems stretching forth rigid yet elastic arms at different heights, the arms of the different stems being so tied and connected together by the intervening fabric that the whole group forms a single self-contained structure firmly rooted into the ground.

125. Details of Column Construction.—Fig. 148 is the cross section of a column 44 centimetres square. The outer portion is built up of six bricks each measuring 22 centimetres long by 11 centimetres wide by 7 centimetres thick, and perforated by eight square holes. The central

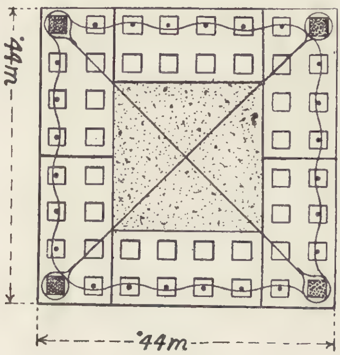


Fig. 148.—Column Section.

core consists of ordinary concrete. Except at the four corners, the vertical reinforcement consists of steel wire, No. 20 French wire gauge, measuring 4.4 millimetres diameter, the wires passing through the holes, which are afterwards filled in with concrete. At each corner of the column the vertical reinforcement consists of a steel bar with an area equal to 43 of the No. 20 gauge wires, and it will be observed that each of these bars practically fills the square hole in the brick. In the horizontal joint of the brickwork steel wires, also of 4.4 millimetres diameter, are woven in and out of the vertical reinforcement as represented in the

section, and at intervals of 70 centimetres, measured vertically, diagonal ties cross from corner to corner of the column, being securely connected to the vertical bars of the reinforcement. The bricks in alternate courses are disposed so as to break joint, and when the nature of the reinforcement is taken into account it is evident that the column is very adequately bonded.

Fig. 149 is a cross section of a column in the crypt

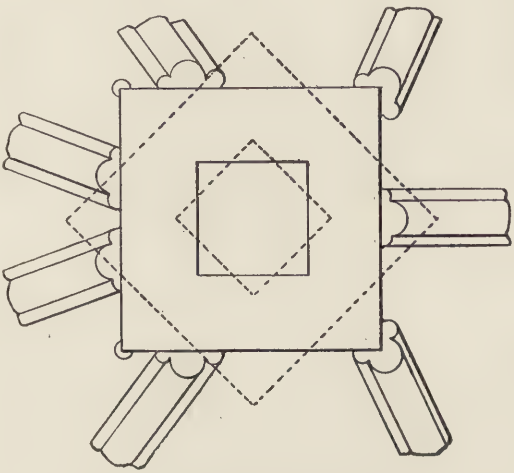


FIG. 149.—Column and Arched Ribs.

showing the projecting ribs. The diagonal square drawn in dotted lines is the outline of the column in the church above.

Fig. 150 is taken from a cross section of the building, and illustrates a portion of the column and floor construction. In this drawing will be seen the curved ribs springing from the columns, and affording rigid support for the church floor. The portions of the upper columns appear to be wider than those in the crypt, this being, of course, because they are placed at a different angle, as explained in Article 124, and as represented by dotted lines in Fig. 149.

Fig. 151 is a plan including four columns, and the curved ribs springing therefrom.

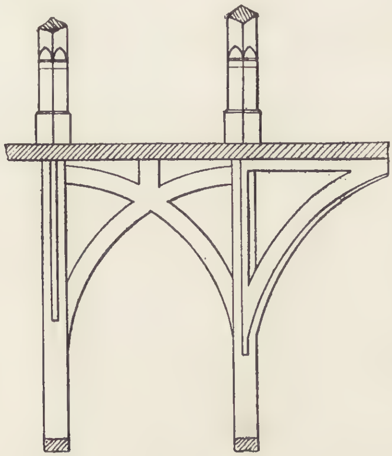


FIG. 150.—Column and Floor Construction.

The twisting of the columns at the floor level of the church is made practicable by the arrangement of the

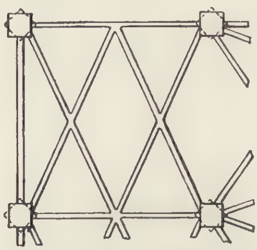


FIG. 151.—Column and Arched Ribs.

vertical reinforcement, in the manner described below, so as to afford support for the overhanging corners of the upper portion of each column.

In the diagrammatic plan (Fig. 152) let aa', $a'a''$, $a''a'''$, $a'''a$ represent the four sides of the crosss ection of a column in the crypt, and bb', $b'b''$, $b''b'''$, $b'''b$ the four sides of its continuation in the church above.

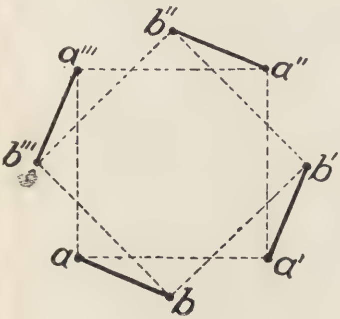


FIG. 152.

Then we have to support four projecting corners, namely, b, b', b'', b'''.

This condition is fulfilled as shown in the perspective diagram Fig. 153, where a, a', a'', a''' are the corner bars corresponding with those in Fig. 148. For the sake of clearness no account is taken in the diagram of any but the four corner bars of the vertical reinforcement.

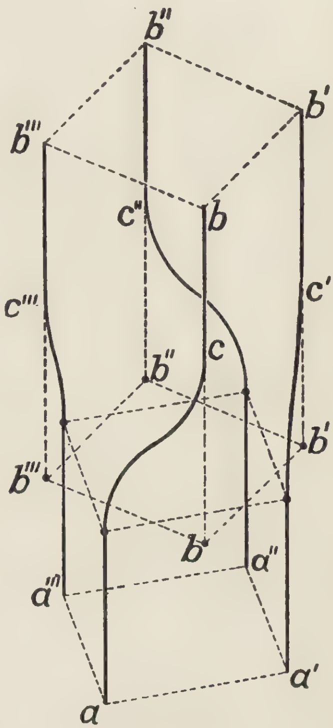


FIG. 153.

The projecting corner b of the upper part of the column is supported by continuing the vertical bar a, as shown in Fig. 153, first in the curved direction $a\ c$, and then vertically to b, which stands for any point in the column vertically above c. The vertical bar a' is continued in a similar manner to c' and b', the bar a'' to c'' and b'', and the bar a''' to c''' and b'''.

The result is shown in plan by the thick lines in Fig. 152, where the axes of the vertical bars are diverted thus—a to b, a' to b', a'' to b'', and a''' to b'''

Assuming that the four bars have been twisted in this manner, let four other bars, cb, $c'b'$, $c''b''$, $c'''b'''$, be attached (see Fig. 153), one at each of the points c, c', c'', c''', and carried vertically downwards, as shown by dotted lines in the diagram, to the lower points b, b', b'', b'''. It is evident that these overhanging points must receive such support as the column above is able to afford, by means of the four bars which form a part of its system of reinforcement throughout its whole height above the floor of the church.

As the octagonal portion of the upper column (see Fig. 150) receives adequate support from below, and as a system of reticulated reinforcement (see Fig. 148) permeates the substance of each part of the column and passes unbroken from one portion to the other, there can be little doubt as to the value of the support given to the projecting corners.

We must remember also that the bars ab, $a'b'$, $a''b''$, and $a'''b'''$, even though they are bent, possess sufficient rigidity to resist a considerable downward force exerted in the directions bcb, $b'c'b'$, $b''c''b''$, $b'''c''b'''$. Moreover, all the vertical rods or wires in the column are also bent to follow curves similar to those of b to c, b' to c', b'' to c'', and b''' to c''' in Fig. 153.

But the projecting corners of the columns in the church receive further support from three other parts of the structure, namely, the reinforced floor slab, the horizontal floor ribs, and the ribs springing from the columns in the crypt, as shown in Figs. 149 to 151.

Some of the longer ribs radiating from the columns are prolonged to form the joists of the church floor, while the shorter ones afford support by means of blocks placed at the point of intersection (see Fig. 150).

As these stiffening ribs are braced together laterally, a series of four-legged frames is formed, each of which is able to carry a very great load upon its centre if the ribs be properly connected at the point of convergence. This part

of the construction is, in fact, based upon the principles governing the design of domed or vaulted structures.

126. Floor Construction.—The design of the church floor—or, as it may be termed alternatively, the crypt roof—is made clear by Fig. 154.

Here the points A, B, C, and D stand for columns. Arched ribs A*i* and A*b* spring from A; similar ribs B*i* and B*a* from B; similar ribs C*i* and C*d* from C; and similar ribs D*i* and D*c* from D. Connecting ribs *ag*, *dg*, meeting at *g*, and corresponding ribs *bh*, *ch*, meeting at *h*, complete the triangulated panel.

Since all these arched ribs are braced together by the concrete-steel floor slab, a comparatively small percentage

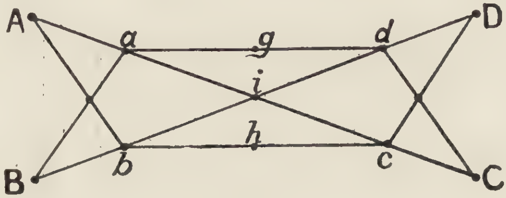


FIG. 154.

of reinforcement is sufficient to ensure very great resistance.

The principle here outlined has been extended to the whole of the floor system, and the result is the complete triangulation represented in Fig. 155, which is a skeleton plan of the ribs 30 centimetres deep, with an average thickness of 7 centimetres below a portion of the floor slab of the church.

The reinforcement of the floor slabs is disposed in meshes, which vary in size in accordance with the diagram of bending moments for different parts. In this way the metal is utilised in the most economical manner, and the dead weight of the floor is reduced to a minimum.

This floor, in which panels measuring 11.50 metres by 7.25 metres are entirely supported upon four piers of 44 centimetres square, is certainly of much interest, not merely

because of its daring design, but also for the economy of construction which it exemplifies.

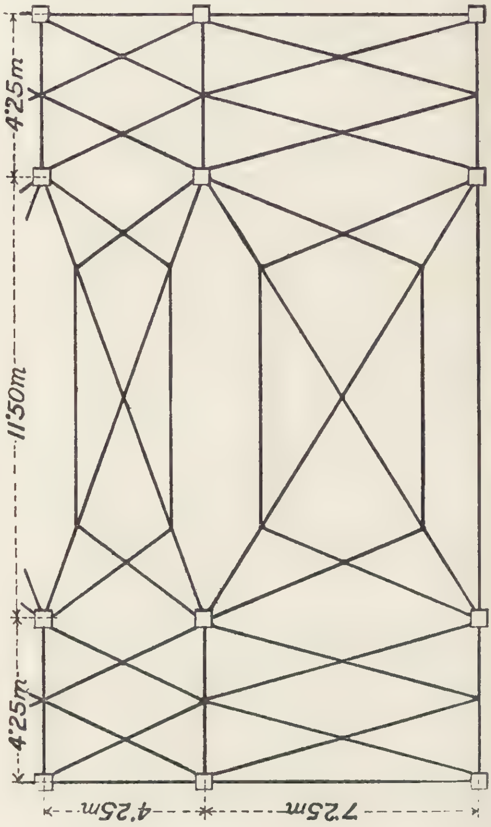


FIG. 155.—Part Plan of Columns Arched Ribs supporting Church Floor.

127. Wall Construction.—As stated in Article 122, the walls of the church are of reinforced brickwork 11 centimetres thick. They are reinforced as described in Article 111, the amount of reinforcement being proportioned according to circumstances.

One of the most remarkable features of this church is the wall construction at an angle of the building where it was necessary to form a passage below the crypt. Here, as shown in Fig. 156, the triangular portion ABC of the building, with a total projection of 8 metres, has no apparent support beyond that of a 44-centimetre square column.

This part of the structure, particularly struck the engineers

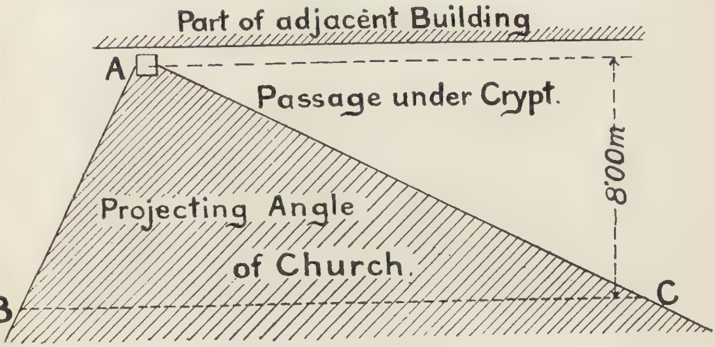


FIG. 156.

of the Ponts et Chaussées who came to inspect the work. On approaching the span AB across the passage, 8 metres in width, they looked for the girder supporting the superstructure, and were considerably surprised when told that, apart from the wall itself, the only member in the nature of a girder was a steel bar 40 millimetres wide by 9 millimetres thick.

Nevertheless, the explanation is perfectly simple. The wall is really a girder 26 metres deep, and therefore abundantly able to span an opening of only 8 metres in width.

CHAPTER VII

Railway Station Dome, Antwerp—Locomotive Depôt, Jura-Simplon Railway

RAILWAY STATION DOME, ANTWERP

128. General Construction.—An interesting example of concrete-steel construction is afforded by this dome, which springs from the roof level of the new Central Railway Station, Antwerp, at 130 ft. above the ground, and rises to the farther height of 130 ft. including the campanile. This detail of the station building was originally intended to be of stone, but as it was found that the foundations would not carry the weight involved concrete-steel construction was substituted, and further reduction of weight has been secured by building the dome with hollow walls. The work was executed throughout by M. Vasaune of Brussels.

As shown in Figs. 157 and 158, the dome comprises four large arched windows placed upon the sides of a square, and upon these rests the dome proper, which in turn supports a campanile. Each window is in the form of a gallery, with seven arcades surmounted by a semicircular arch of 32.8 ft. radius. The arches are framed by an archivault, 11.5 ft. high, receiving at its periphery the haunches of the dome.

The entire structure, which weighs 1,800 tons, rests upon the four columns at the angles of the windows, these being the only points where solid support was obtainable.

The columns are Y-shaped in cross section, and at the height of the centres of the arches they are subdivided into three separate members. The tail of the Y is extended in the diagonal plane in the form of a thrust block rising obliquely between the two shells of the dome.

In the horizontal plane passing through the tops of the archivaults is a member in the form of a flat ring, 4.92 ft. wide, which is supported at eight equidistant points by the

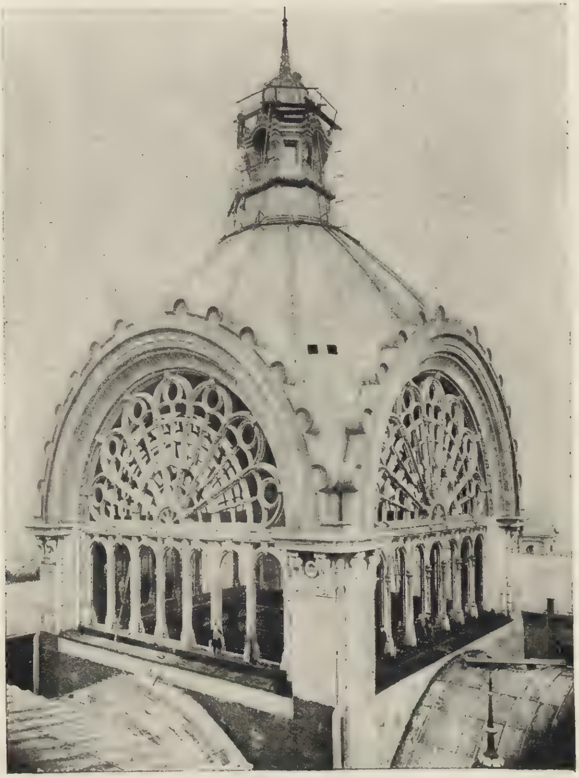


FIG. 157.—Railway Station Dome, Antwerp.

tops of the archivaults and the ends of the thrust blocks. This member serves two purposes, balancing horizontal reactions due to the obliquity of the thrust blocks, and resisting tensile stresses due to the ribs of the dome. Tie-

rods, commencing at the archivault, go down in the mid-ribs of the arches, and extend into the two central columns of each gallery of arcades for supporting a horizontal beam, hidden in the entablature of the gallery, by which the upper lights are carried.

The dome consists of two superposed shells, separated by a distance varying from 3.28 ft. to 6.56 ft. The internal shell, which forms the ceiling of the entrance hall, is decorated with sunk moulded panels, which leave only flat bands on the inside of the shell. Some of these bands are formed by a skeleton of beams and trimmers which are supported on the annular ring of the dome, and carry the entire weight of that structure. This skeleton was erected first, and served to support the moulds for the panels, which were next filled in with concrete

The external shell of the dome has a uniform thickness of 3.15 in., and is relieved by six moulded ribs following meridian lines. It is supported upon the internal shell by small distance pieces normal to the two surfaces, this method of support allowing for the unequal expansion of the shell due to the oblique direction in which it encounters the rays of the sun.

129. System of Moulding.—One of the most interesting features connected with the construction of the dome was the ingenious method of moulding devised by M. Vasaune, which was employed in forming ornamental details and imitation sculptures of all kinds. So numerous and so varied were such details that it would have been quite out of the question to make timber moulds for them, especially as much of the work had to be executed on surfaces some curved in one direction and others in two directions.

M. Vasaune first executed in plaster a model of the ornamentation to be reproduced in concrete. He then spread upon this a layer, from about 1 in. to 2 in. thick, of a paste made of magnesium oxychloride and sawdust, which quickly hardens and constitutes a light and strong mould that can be worked like wood. The same plaster model was used several times for the production of moulds.

The construction of the window details was especially difficult because of the great richness of the ornamentation,

and of the exactitude with which the moulds had to be made to ensure the perfect fit of the different sections. Those parts of the windows which form galleries were filled with

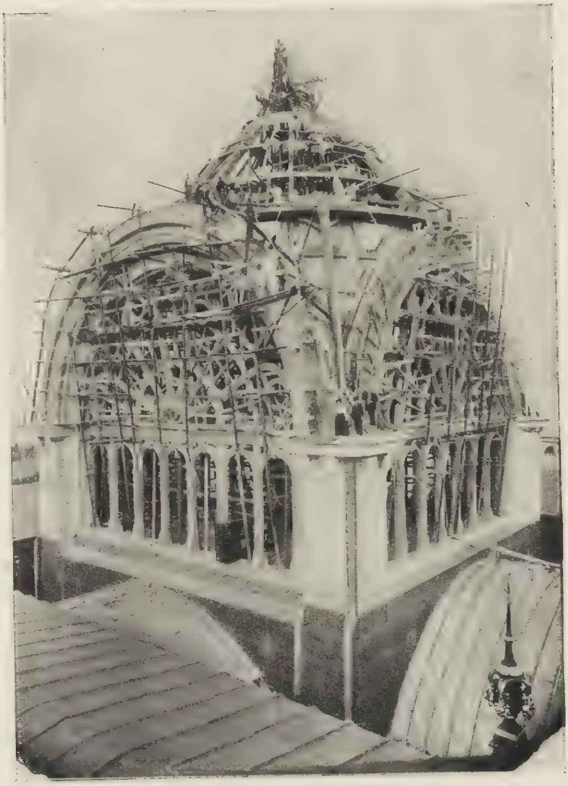


FIG. 158.—Dome under Construction.

concrete at one operation in moulds built up in position on the site. For the semicircular portion of each window a complete mould was set out on a perfectly level platform,

and the mould showed in hollow one face of the entire semicircle.

The concrete was moulded to a thickness of 2 in., and the moulded part was divided into specially marked portions. Then the moulding, followed by the same division into parts, was repeated to obtain the opposite face. In this way two corresponding portions, placed back to back, formed a hollow structure representing one element of the window framing. The parts were afterwards used in building up the framework, just as stones are employed, but steel bars were placed as reinforcement in the hollow spaces, which were filled up with concrete so as to connect the two moulded parts and to form a monolithic structure of great rigidity.

The particulars relative to this work have been taken by permission from the Proceedings of the Institution of Mechanical Engineers.

LOCOMOTIVE DEPÔT, JURA-SIMPLON RAILWAY

130. Types of Locomotive Depôt Design.—A novel application of concrete-steel is to be found in the locomotive depôt recently erected at the station of Rennes on the system of the Jura-Simplon Railway Company, near Lausanne. Our readers are aware that buildings for the accommodation of locomotives are of two distinctive types: (1) roundhouses with turntables giving access to radiating tracks, and (2) rectangular sheds with parallel tracks. The latter type appears to be generally preferred in the present day, and is that adopted in the design of the buildings described in the present chapter.

131. British Practice.—The most recent designs of locomotive sheds are to be found in Great Britain, where, curiously enough, a return has been made to the employment of timber framework, this development at first sight appearing to constitute a movement of retrograde character.

On consideration, however, it must be recognised that there is some justification for the abandonment of metal work, which, owing to oxidation and injury due to the sulphurous fumes contained in smoke emitted from the locomotives, is so rapidly corroded as to involve heavy

maintenance charges, by reason of the necessity for replacing large portions of the work at comparatively frequent intervals.

In fact, the life of a steel roof truss cannot be put at more than ten or fifteen years. On the other hand, the products of combustion, thanks to the empyreumatic substances disengaged therefrom, tend to preserve timber and impart to it in some measure the quality of non-flammability.

The type of locomotive shed favoured in this country offers the advantage of being suitable for the application of flat roofing, which is difficult and costly in the case of a roundhouse. Flat roofing lends itself to more effective lighting arrangements and to the exclusion of cold air, which in cold climates is apt to freeze the water in the locomotive boilers. Further, it can be constructed very economically, for a thin flat roof supported by light columns costs far less than the complicated circular roofing system of a roundhouse.

132. Report by Prof. Bosset.—Having been commissioned by the Jura-Simplon Company to make a careful examination of the various designs of locomotive sheds exemplified by the railways of different countries, Professor Bosset reported in favour of the British type, but with the modification that all parts of the structure usually built of timber should be constructed in concrete-steel. In consequence of this report the State *service du contrôle* decided to reject the two sets of plans previously submitted by the company—the first including steel and the second timber framework—for the depôt required at the station of Renens.

133. Design by Prof. Bosset.—After a minute study Professor Bosset produced the design for a roof system entirely in concrete-steel, including the roof proper, hoods for collecting smoke from the engines in the shed, and vertical flues for discharging the smoke into the open-air. Professor Bosset recognised in the British flue arrangement the following advantages: (1) that when the funnel of a locomotive is introduced into the hood beneath a series of flues it projects to such a distance that at whatever

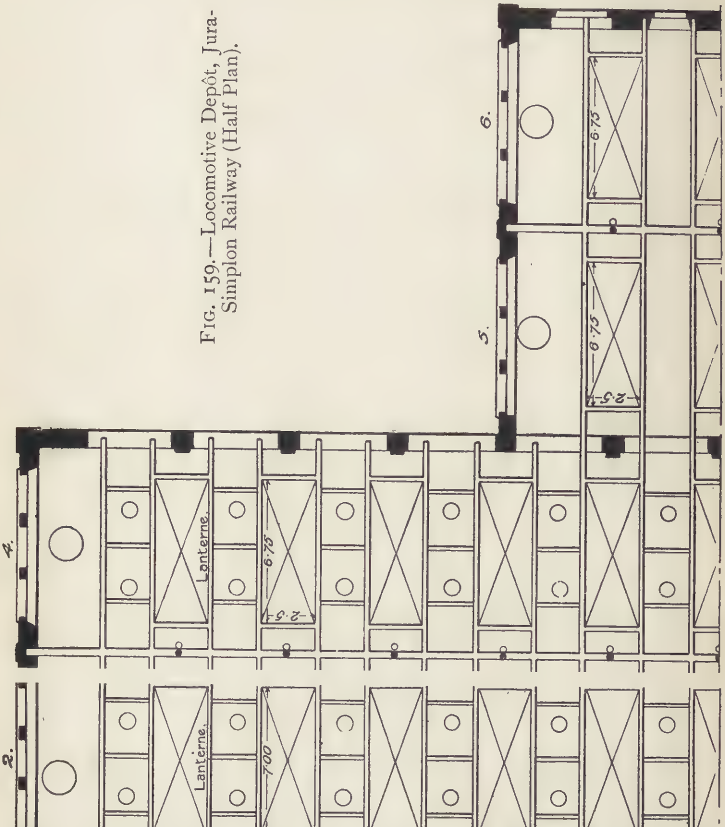


Fig. 159.—Locomotive Depôt, Jura-Simplon Railway (Half Plan).

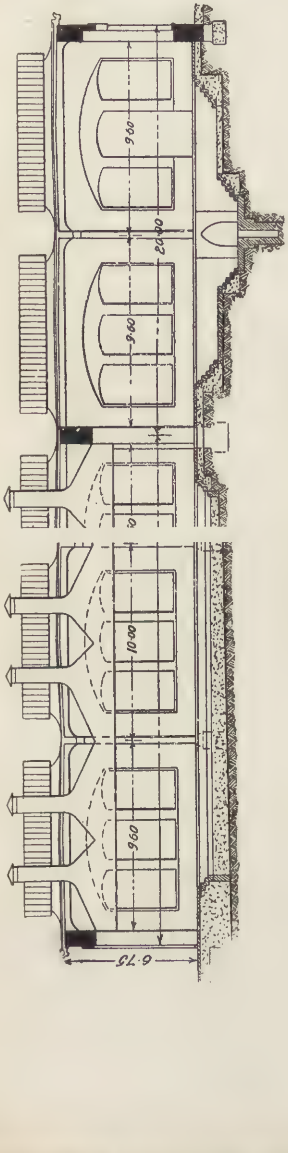


FIG. 160.—Longitudinal Section.

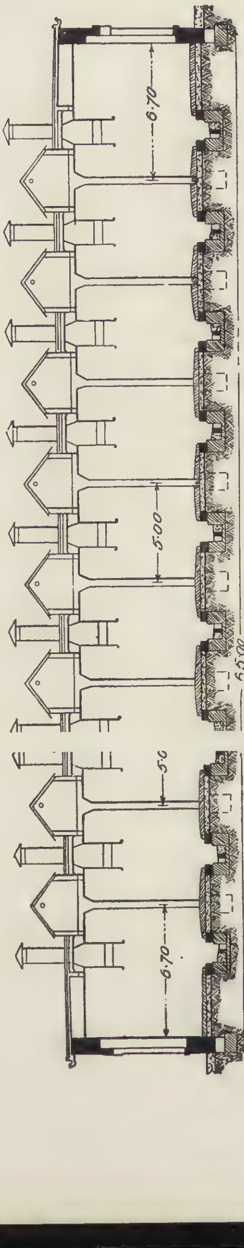


FIG. 161.—Transverse Section.

point the locomotive may be stopped the smoke can always find its way out by one or other of the vertical flues; and, (2) that being fitted with butterfly valves, the outlets can be regulated in very cold weather, so that, while sufficient draught is maintained for carrying away the smoke, cold air is prevented from descending into the building.

The chief difficulty presented was to reproduce this arrangement in concrete-steel. The problem was solved by designing the hoods in reinforced concrete of superior quality, providing for their suspension from the roof by steel bars furnished with turn-buckles to permit adjustment. The vertical flues, also of reinforced concrete and fitted with butterfly valves and finished with weather cones, were designed to fit into sockets as described in Article 138. Full details relative to the structural features of the work will be found in the succeeding articles.

134. General Construction.—The roof proper consists of a series of beams supported by columns of concrete-steel. Fig. 159 is a half plan showing the six bays formed by the main beams, the panels into which it is divided by the secondary beams, and the positions of the lanterns and the smoke flues. The supporting columns are spaced 10 metres apart, the height of these being purposely kept down to 6 metres, so as to do away with unnecessary space and thereby to make more easy the maintenance of an equable temperature. Except where the lanterns occur the roof is flat, being composed of a concrete-steel slab supported by beams, as in the case of a concrete-steel floor. The roof surface has slopes of about 1 in 30, but these were varied as necessary to ensure the flow of water towards the columns, against which rain-water pipes are fixed. It was originally the intention to utilise the interior of the columns instead of separate drain pipes, but the *service du contrôle* refused to sanction this arrangement as being contrary to precedent. Other structural features of the building are illustrated by Figs. 160 and 161.

135. Roof Slab.—The flat roof slab, with an area of about 2,670 square metres, is supported along the edges on a thick bed of sand—covered by a layer of paper—laid on

the upper surface of the walls, the expansion joint so constituted being intended to permit lateral expansion of the concrete-steel without involving the risk of injury by

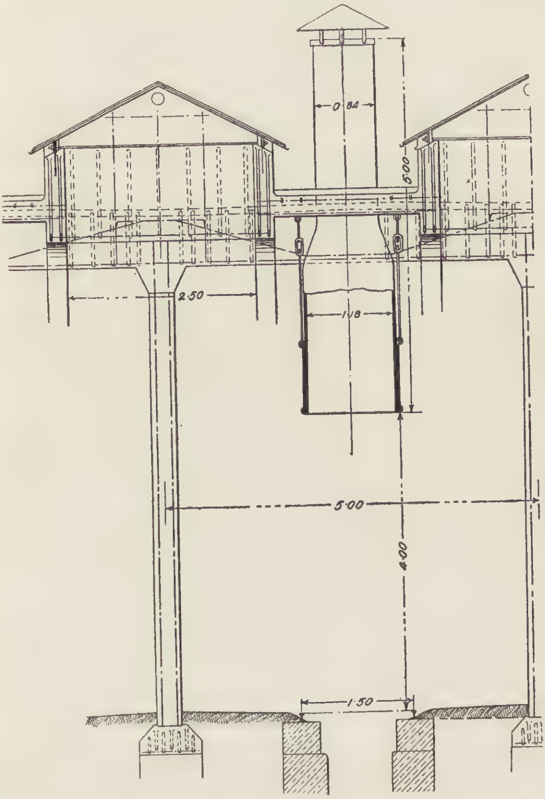


FIG. 162.—Lanterns, Smoke Hoods, and Flues.

dilatation of the slab. This detail of construction has the effect of detracting in some measure from the monolithic character of the building as a whole, and was insisted upon by the engineers of the control service. It is by no means

certain that the expansion joint so designed actually answers its intended purpose. The idea was that the grains of sand would act after the manner of balls in a bearing. But as grains of sand are neither smooth nor perfectly spherical like steel balls, it may be doubted whether the action intended by the designer has been realised in practice.

The surface of the slab is covered with material termed "ciment de bois," over which is spread a layer of gravel 10 centimetres thick, to protect the cement from the direct rays of the sun.

"Ciment de bois" is composed of a layer of millboard impregnated with bitumen, and three layers of paper similarly treated, and on this roof it was prepared as follows: The millboard was first laid upon a thin bed of sand spread upon the surface of the roof, the object of the sand being to enable the millboard to expand freely under the influence of heat; next, the upper surface of the millboard was covered with bitumen, applied hot with a brush, and on this was spread the first sheet of paper. This and the remaining sheets were covered with bitumen, as in the case of the millboard, with the result that the "ciment de bois" included four layers of bitumen, one layer of millboard, and three layers of paper.

136. Lantern Frames.—Fig. 162 illustrates the construction of the lantern frames. At the sides of these precautions were necessary to prevent the penetration of moisture. Strips of zinc were interposed for a distance of 15 centimetres between the millboard and the paper of the "ciment de bois," and cemented by means of bitumen applied with a brush, so that the zinc strips practically became extensions of the bituminous roof covering. Each strip was bent upwards for a height of from 15 to 20 centimetres, to form flashing against the side of the lantern framing.

137. Gutters.—The gutters of the roof were constructed in reinforced cement, covered with sheet zinc laid in such manner as to permit free expansion under the influence of heat, and the gutters were bordered at the edge by a light curb, to maintain the layer of gravel on top of the roof at the thickness of 10 centimetres.

The question may be asked whether it would not be possible to make watertight gutters of concrete-steel without the employment of a zinc lining. In reply it should be

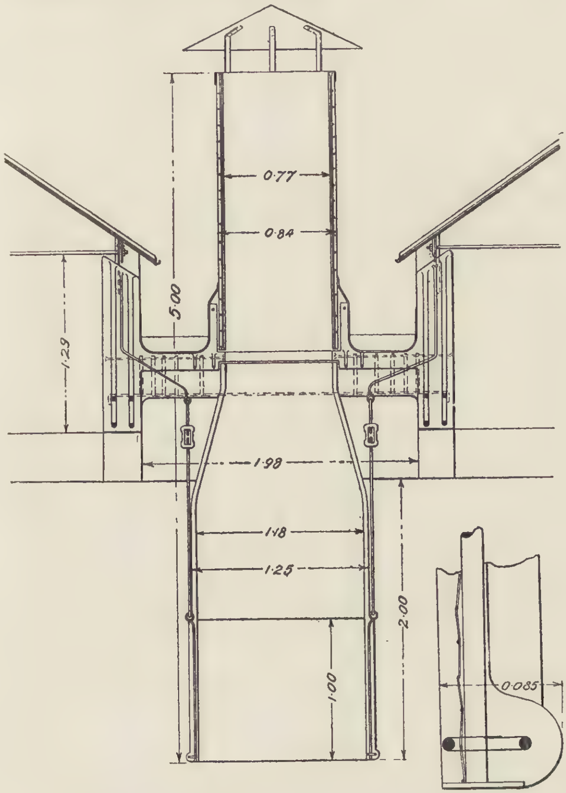


FIG. 163.—Details of Smoke Hood and Flue.

said that, while gutters of comparatively small length can be so constructed without risk of leakage, it is certainly preferable to add a lining to gutters of considerable length. In the locomotive depôt at Renens some of the gutters are

65 metres long, and if not lined it would be scarcely possible for them to remain permanently watertight, because if one or two cracks were produced by expansion and contraction the whole length must be regarded as leaky.

To render the cement guttering and the zinc lining capable of independent action under the influence of temperature changes the following method was adopted. After the channel had been moulded, concrete was added where necessary to ensure the proper fall; then the inside of the gutter was covered with bituminised paper, over which the zinc lining was laid. The employment of paper was thought desirable, because some brands of cement contain an excess of free lime, and should not be in direct contact with the zinc.

138. Smoke Hoods and Flues.—Fig. 163 shows the details of the smoke hoods and flues. The hoods are formed of concrete-steel, in plates 35 millimetres thick.

The hood descends to such depth that the upper extremity of a locomotive funnel is about 40 centimetres above the lower edge of the hood, which fits into a socket and is suspended by steel bars, provided with the turnbuckles above which the bars are bent outwards and continued into the concrete of the lantern walls, where they are securely anchored, as shown in Fig. 163.

The vertical flues, also of concrete-steel, are circular in cross section, with an internal diameter of 75 centimetres, the wall of the flue being 35 millimetres thick. Each flue was fitted in a socket of concrete-steel moulded upon, and monolithic with, the roof slab. Thus the flues are not immovably fixed to the roof, but are free to slide within the sleeve joints, a condition very necessary for the avoidance of undue strain upon the concrete of the roof and the chimneys. The socket projecting above the roof slab is formed of concrete-steel coated with tar, over which is zinc flashing, connected with the "ciment de bois" in the manner previously indicated.

CHAPTER VIII

A French Villa

139. General Description.—Bourg-la-Reine, a modern village of some 3,500 inhabitants, is a favourite summer resort about three miles along the Orleans Road on the southern side of Paris. In this place is situated the villa built for M. François Hennebique, in accordance with the well-known system of construction with which his name is identified. The site of the house and garden covers an area of nearly 25 metres square, the eastern and southern sides of which are bounded by the Avenue Victor Hugo and the Avenue du Lycée-Lakanal respectively.

The house formerly standing on this plot of land was a long narrow building, and with its outbuildings extended along the whole frontage on the last-named thoroughfare. Built of the volcanic stone known as tufa, architectural character had been neglected in its design, as much as the comfort and health of its occupants.

The new structure, erected at the north-east angle of the site, and with the principal façade on the Avenue du Lycée-Lakanal, suffers from too much architectural character. It is one of those weird constructions from which, with all our defects, we have been spared in England, except perhaps in outdoor exhibitions and at some popular watering-places. Viewed from the street, the villa presents an appearance something like that of a pier pavilion with a promenade deck on top, and from its midst springs a water-tower, which, rising to the height of 40 metres above the road level, is suggestive of a lighthouse combined with the fighting top of a modern warship, as may be gathered from the view reproduced in Fig. 164.

So far as internal arrangements are concerned, however,

there is no ground for criticism. The house is admirably planned; spacious corridors and balconies connect the

FIG. 164.—A French Villa (Garden Front).

rooms of the principal floors; terrace and roof gardens, conservatories and towers provide ample space for flowering

plants; and special attention has been devoted to means of access for light and air.

140. Levels.—The gradient of the Avenue du Lycée-Lakanal from the corner of the Avenue Victor Hugo is about 1 in 14, in consequence of a cutting made in the direction of Bourg-la-Reine Railway Station, but the level of the garden remains unaltered, the earth being held up by a retaining wall. This peculiar condition of the site presented some difficult problems for solution, among them being that of providing convenient means of access to the coach-house of the new building. Again, the rain-water drains, as well as those of the kitchen and other domestic offices of the old house, formerly discharged into a well, or catchpit, on the adjoining property. Thence the water flowed to the street gutter, which it was compelled to follow for a distance of about 350 metres, because no drains or sewers had been provided in the main road. This state of things, as may readily be imagined, was not entirely appreciated by residents in the immediate neighbourhood.

After careful consideration the designer decided that the *rez-de-chaussée*, or ground floor, of the new villa should be level with the road surface of the Avenue Victor Hugo, and that the *sous-sol*, or basement floor, 3 metres below, should correspond in level with the footpath of the Avenue du Lycée-Lakanal.

141. Basement.—The various rooms and domestic offices in the basement comprise the servants' hall and bedrooms, living rooms for the gardener, a lodge or lobby for the concierge, a motor-car garage, cellars for wine in casks and bottles, coal and wood cellars, provision and fruit stores, a hot-water boiler-room, workrooms, and a photographic laboratory. A private entrance opening from the Avenue du Lycée-Lakanal is situated conveniently for access to the railway station. This entrance opens into a lobby communicating on one hand with the passage leading to the storerooms and wine cellars, on the other with the room of the concierge, and is at the foot of the stairway lettered *Entree* in Fig. 165.

The storerooms and cellars are beneath the hall and

children's rooms on the ground floor. They are almost entirely under ground, for on the garden façades the level

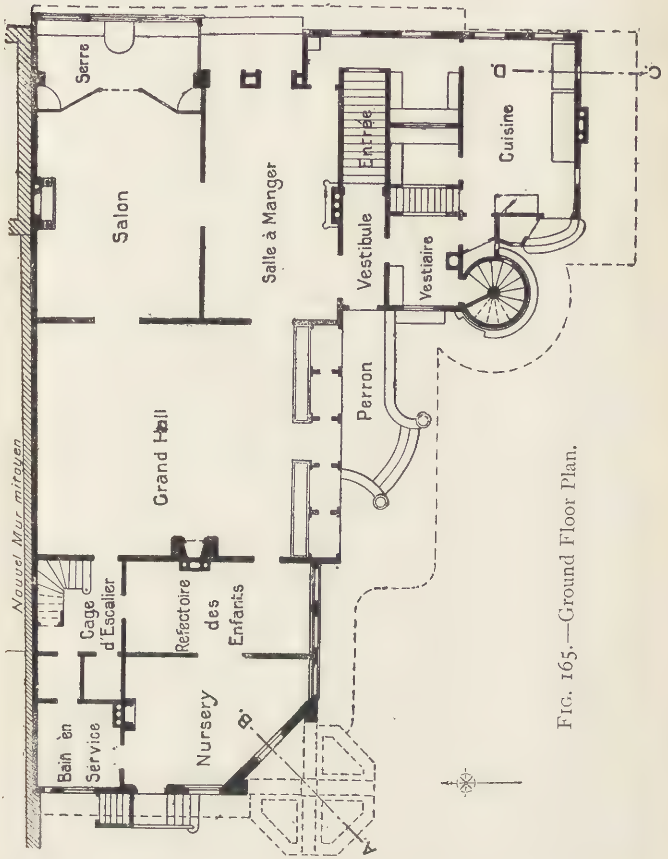


FIG. 165.—Ground Floor Plan.

of the ground has been raised by earth excavated for the construction of the foundation and the basement.

The old party wall, indicated by broken lines in section

b, Fig. 166, was demolished and rebuilt in stone, the position of the new wall, *nouvel mur mitoyen*, being shown in Fig. 165. The heating chamber, which contains a hot-water boiler, is placed near the base of the water tower, and is provided with alternative means of access, one from the outside by way of the tower staircase, and the other from the inside by a lobby at the foot of the service staircase. Thus stoking can be performed from the outside without communication with the house, or from the interior as may be most convenient.

The flue of the hot-water boiler may be observed close to the water tower in Fig. 165.

Fig. 166 contains sections of the concrete-steel walls and foundations. *a* is a section through the wall of the forecourt below the balcony of the dining-room; *b* is a section through the northern wall of the wine cellar, showing bins on the inside and the old party-wall on the outside; *c* is a section through an inner wall separating two of the cellars; and *d* is a section through the wall of the concierge's room.

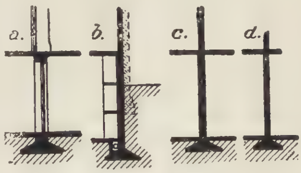


FIG. 166.

With the exception of the party-wall the whole of the foundations, walls, and other details in the basement are of concrete-steel.

142. Ground Floor.—Two entrances give access to the principal rooms of the villa. The chief entrance, or that used by visitors, is reached by a carriage-way leading from the Avenue Victor Hugo through the garden and up to the *perron* outside the doorway of the hall. This apartment, with a length of 9.50 metres and a width of 8 metres, opens upon the drawing-room and dining-room on the right hand, and upon the children's dining-room and the main staircase on the left hand. Views of the hall and dining-room will be found in Figs. 167 and 168. The second entrance, intended for use by the family, is at the head of the staircase rising from the Avenue du Lycée-Lakanal to a vestibule which opens into the dining-room on one side, the cloak-

room on the other, and leads also to the *perron* outside the hall.

FIG. 167.—View of Hall

FIG. 168.—View of Dining-Room.

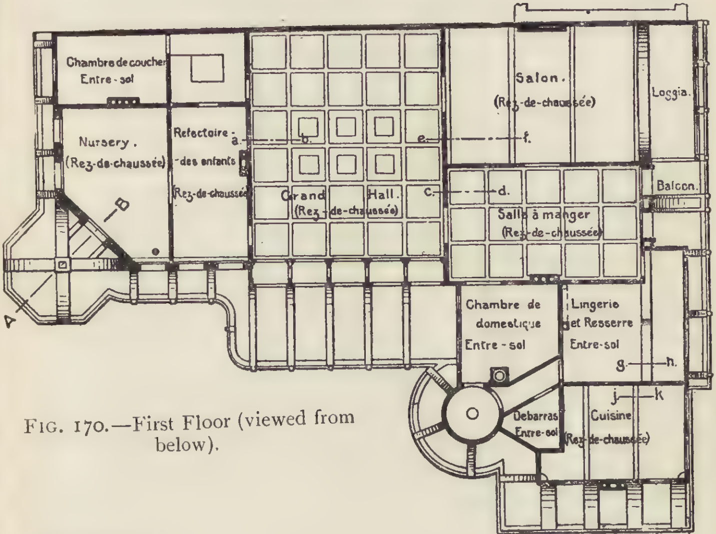


FIG. 170.—First Floor (viewed from below).

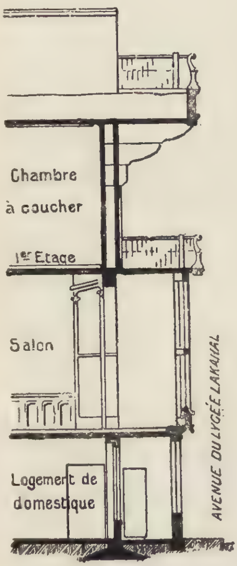


FIG. 169.

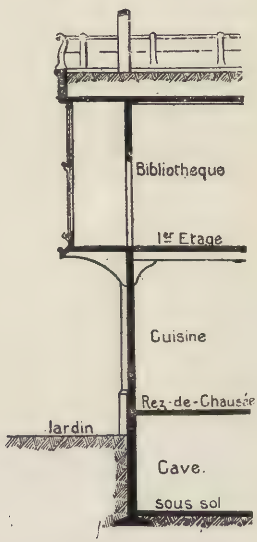


FIG. 171.

Ventilation of the entire suite of rooms on the ground floor is effected by various openings and air ducts, in addition to the windows and doors. The verandah conservatory outside the drawing-room (see Figs. 165 and 169) faces the east, and is completely separated from that room by a glazed partition with a door at either end and a large sliding sash in the middle. One end of the conservatory is connected with the open balcony outside the dining-room, a current of air being established between the balcony and the north wall of the villa, which projects beyond the general façade. These structural features provide excellent means for regulating the temperature of the drawing-room, to which ample light is admitted through the glazing of the conservatory.

Fig. 165 shows the position of the kitchen, which, being 5 metres in height and exposed on three sides to the outer air, can be very efficiently ventilated so as to prevent the smell of cooking from reaching the dining-room and other parts of the house.

It should be mentioned that the upper part of the kitchen is separated from the dining-room by a linen-store on the mezzanine floor, beneath which there are a service-room and a corridor affording communication between the kitchen and the dining-room. This passage is formed with a floor of concrete-steel extending over the entrance lobby at the foot of the private entrance in the Avenue du Lycée-Lakanal.

On the western side of the hall the children's dining-room, nursery, and bathroom constitute an entirely separate suite of rooms, and yet are quite easy of access. The nursery is well lighted, and at the same time shielded from the direct rays of the sun by the octagonal tower and the balconies of the first storey (shown in broken outline in Fig. 165). A stairway leads from the nursery directly into the garden. All the principal rooms on the ground floor have the uniform height of 5 metres.

With the object of preserving as far as possible the previously existing levels of the garden, and to avoid the destruction of trees, it was decided to make the nursery of the form shown in Fig. 165.

Ample shade is provided along the western and southern

façades by the balconies, galleries, and tower indicated in broken outline in Fig. 165, and to some extent by the sections in Figs. 169–171.

The broad projecting terrace on the garden front shields the principal entrance from the direct rays of the sun. The concrete-steel uprights which carry the corbelling are of double-T section, as shown in plan by Fig. 165, and have a thickness of 11 centimetres, the recessed part of the uprights serving to receive the hinged window-sashes and shutters, which remain open throughout the summer, thus virtually transforming the hall into an interior court, from which the whole of the house is abundantly supplied with fresh air.

143. First Floor and Entresol.—Fig. 170 is a plan of the first floor viewed from below, and on which are indicated, for the purpose of identification, the names of the rooms immediately below the floor in question. The *loggia* and the *balcon*, as lettered, are at first floor level, being above the verandah conservatory and the balcony shown in Fig. 165.

It will be seen by Fig. 170 that the floor above the hall is divided into thirty square panels, each 1.60 metres square, by a series of longitudinal and transverse beams of tapered cross section (see section *ab*, Fig. 172). The beams are 20 centimetres deep by 17 centimetres wide at the top and 12 centimetres wide at the bottom, but those at the walls, being tapered on one side only, measure 14.5 centimetres wide at the top, the floor slab is 10 centimetres thick. In six panels over the middle of the hall ceiling lights of armoured glass 70 centimetres square are inserted, as indicated in the plan.

Fig. 170 also shows the corbelling, with a projection of 3.40 metres, supporting the balcony over the principal entrance to the villa.

The floor above the dining-room is divided into fifteen square panels of the same dimensions as those in the hall. Support is given at the walls by tapered beams measuring 14 centimetres wide at the top and 12 centimetres wide at the bottom, while the intermediate beams are 16 centimetres wide at the top and 12 centimetres wide at the bottom, the

projection below the floor slab in each case being 16 centimetres. The thickness of the floor slab itself is 6 centimetres (see section *cd*, Fig. 172).

The floor above the drawing-room is divided into three panels each 5.70 metres long by 2.50 metres wide. The outer panels are supported by rectangular beams (see section *ef*, Fig. 172) projecting from the walls, the dimensions being 18 centimetres wide and 25 centimetres deep below the floor, and the intermediate beams are of the same dimensions.

Fig. 170 also shows the supports of the balcony, with a floor 10 centimetres thick extending from the kitchen to the north gable of the house. Part of this balcony is supported by corbelling, of which the under side is shown in

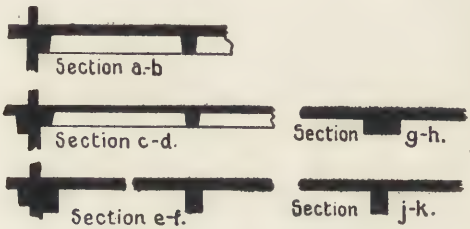


FIG. 172.—Floor Sections (see Fig. 170).

the drawing. The part lettered *loggia* is over the verandah outside the drawing-room, and consists of a concrete-steel floor supported along one side by the front wall of the house, and on the other side by the uprights of the verandah, as illustrated in Fig. 169. Fig. 172 (section *gh*) shows the girder, 40 centimetres wide by 15 centimetres deep, beneath the linen-room on the entresol, the floor slab of this room being 10 centimetres thick. The other two rooms on the same level have floors 12 centimetres and 8 centimetres thick respectively, without any intermediate support. The floor above the kitchen is divided into three panels, for which intermediate support is afforded by two concrete-steel beams each projecting 25 centimetres below the floor slab, and having a width of 18 centimetres (see section *jk*, Fig. 172). The floor slab is 8 centimetres thick, and the reinforce-

ment of the entire construction is connected with that of the walls.

The balcony outside the southern wall of the kitchen is carried by four massive brackets, and has a floor slab 10 centimetres thick. It will be seen in Fig. 170 that this balcony is connected with the segmental gallery extending partly round the water tower, the latter construction being carried by three cantilever brackets and having a floor slab 10 centimetres thick.

Near the water tower will be observed a large flue, which is that proceeding from the hot-water boiler in the basement. This, as in the case of all the other flues indicated in the plan, is constructed of reinforced concrete, monolithic with the walls of the building.

The floors above the nursery and children's dining-room consist of single slabs of concrete-steel 14 centimetres thick, and the adjoining bedroom on the entresol has a similar slab 10 centimetres thick. On the west side of the nursery wing the upper balcony is carried by four cantilevers projecting from the main walls. The balcony on the south side of the nursery is supported by three cantilevers.

Fig. 173 is a plan of the first floor of the villa. Communication between the various rooms on the eastern façade is conveniently afforded by the wide balcony extending almost along the whole front of the building.

It is worthy of note that, with the object of minimising the effects of extremes of temperature, the walls along this façade were built hollow, as shown in the plan and also in the section, Fig. 169. The same method of construction has also been applied to the nursery wing. The study situated immediately over the kitchen opens into the *palmarium*, which is a glazed verandah communicating by way of the vinery with the open-air terrace garden, which occupies a central position on this storey.

The balcony of the terrace garden gives convenient outside access between the octagonal *salon* in the children's wing and rooms in the other portion of the house, while communication is also afforded by the covered gallery which runs across the inner end of the terrace garden.

144. Octagonal Cantilever Tower.—In Fig. 173 will

be noticed a small *salon* above the angle cut off from the nursery. This room occupies the lower portion of an octagonal tower, the construction of which is illustrated in Figs. 174 and other drawings, while the general appearance of the work is shown by Fig. 175. The tower weighs 180 tons, and projects about 4.50 metres from the wall of the main building, without taking into account the circular turret

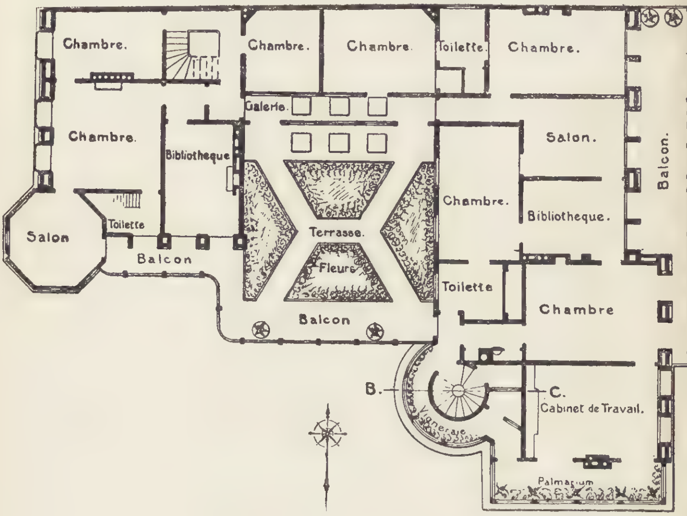


FIG. 173.—First Floor Plan.

near the top. The corbelling is formed by two cantilever brackets intersecting at the axis of the tower, which is further supported by monolithic connection with the walls of the villa and the anchorage of its reinforcement into the concrete of the latter (see Fig. 176).

Rising above the level of the terrace garden on the roof of the main building, the upper portion of the tower affords space for a kiosk, the flat roof of which is covered with a bed of earth 1 metre deep, in which fruit trees are planted (see Fig. 177). A wide exterior balcony of concrete-steel

extends round the top of the tower, which forms a very pleasant retreat in summer-time. A staircase, giving access to this little terrace, is provided in the turret built on the exterior face of the tower. The tower represents a total dissymmetrical load of nearly 200 tons.

145. Roof.—The flat roof at the top of the house is covered with earth to the depth of 1 metre, so that vegetables and flowers may be grown under favourable circumstances, and to afford an adequate depth for the roots of shrubs and trees. Taking the weight at earth of 80 lb. per cubic ft., the metre depth of soil in the present case involves a dead load of 1,280 kilogrammes per square metre, to say nothing of the weight of a hothouse, which is situated along the northern wall of the building.

146. Water Tower.—The most striking feature of the villa undoubtedly is the water tower, to which incidental reference has already been made. The tower was built for the primary purpose of supporting a reservoir with a capacity of 25 cubic metres at a height of 18 metres above the ground floor level, or 8 metres above the roof of the house.

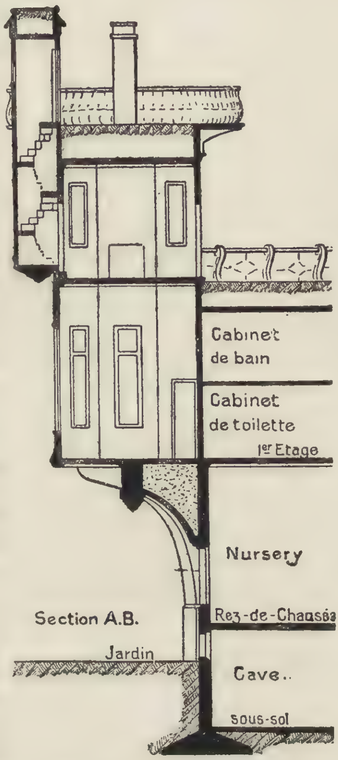


FIG. 174.—Octagonal Tower (section AB in Fig. 170).

The erection of a storage tank at this elevation was necessary to provide water at adequate pressure during the

day, as supplies direct from the town mains are only available during the early hours of the morning. The

FIG. 175.—View showing Octagonal Tower.

tower also serves the purpose of providing convenient means of communication between the basement and the

different floors of the building, as may be seen by reference to Figs. 165, 170, and 173.

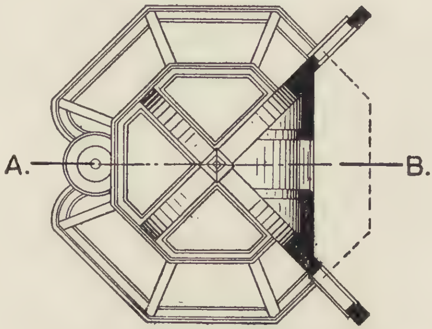


FIG. 176.—Supports of Octagonal Tower.

The top of the reservoir has been utilised as a balcony garden, and by continuing the tower still higher a second gallery and a small observation chamber were placed at the disposal of those desiring to enjoy a view of the surrounding

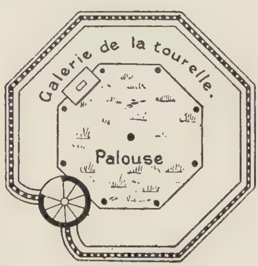


FIG. 177.—Top of Octagonal Tower.

country and to obtain fresh air far above the neighbouring houses.

In the design of the tower the engineer has seized the opportunity of demonstrating the elasticity and homogeneity of concrete-steel, and from our present point of view the

success of his demonstration makes amends for the somewhat incongruous appearance of the construction as an architectural detail of the villa.

Fig. 178 is a vertical section of the tower through the line BC in Fig. 173. The total height from the foundation to the top of the vane is 40 metres, and, as may be seen by the drawing, the structure consists of two parts, one fitted into the other something like the joints of a fishing-rod.

The internal diameter of the lower portion is 2.50 metres, the thickness of the walls being 11 centimetres, except near the bottom, where a slight batter is established in order to permit the thorough connection of the tower with the lower walls of the villa. The tube, of 2.50 metres diameter, continues up to the height of 3.50 metres above the terrace garden, and a few centimetres above the level of the latter the smaller tube commences, this portion having an internal diameter of 1.20 metres and a wall thickness of 5 centimetres. The socket formed by the overlap of the lower tube provides for the doorway which gives access to the terrace.

The treads of the staircase in the lower tube are of concrete-steel, moulded in advance with a diagonal plate on the under side of each, so that when fixed in juxtaposition the under sides of the steps formed a continuous ceiling. The treads were embedded to a depth of 3 centimetres in the concrete of the tubular wall. No string course was required at the end of the treads, which terminate so as to form a central well-hole of 50 centimetres diameter. Up to the level of the entresol this well-hole is filled by a concrete-steel tube, as shown in the section, and upon the closed top of the tube a lamp standard is fixed.

The most interesting part of the tower is that above the terrace garden. As represented in Fig. 178, the staircase of the upper portion commences within the prolongation of the 2.50-metre diameter tube. In addition to the connections between the two tubes by horizontal members just above the terrace level a secure fixing is furnished by the annular diaphragm of concrete-steel near the upper end of the larger tube. This diaphragm is utilised as the

bottom of a receptacle for earth in which creepers and flowering plants can be grown.

Further, four radial buttresses, of concrete-steel, of which two may be seen in Fig. 178, are fixed outside and incorporated with the prolongation of the lower tube, above which they are carried inwards to stiffen the smaller tube, and continued up to the bottom of the water reservoir.

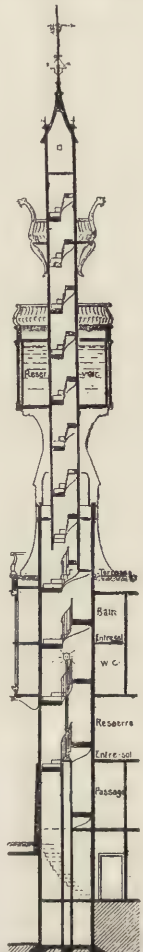


FIG. 178.—Water Tower.

The hollow shaft of the tower, with the reduced diameter of 1.20 metres and walls of 5 centimetres thick, passes through the centre of the reservoir to give access to the upper balconies and observation chamber. A horizontal section of this shaft, taken immediately above the concrete-steel cover of the tank, shows a ring of 1.20 metres diameter inside and 1.30 metres diameter outside, stiffened by four pilasters of 20 centimetres width and 5 centimetres projection, the sectional area of concrete-steel being 2,364 square centimetres. A horizontal section taken immediately above the lower portion presents the same area of 2,364 square centimetres plus four exterior counterforts of 20 centimetres width and 30 centimetres projection, making a total area of 4,764 square centimetres. A horizontal section taken at the height where the counterforts are of minimum thickness gives an area of 3,960 square centimetres.

The construction of the upper portion of the tower can be very well defined as that of a tube fixed in a vertical position, having a height of 22 metres, an inside diameter of 1.20 metre, and an outside diameter of 1.30 metre, on which has been threaded and fixed about half-way up, upon four exterior stays, an annular

reservoir weighing about 45,000 kilogrammes, and above this a second annular ring represented by the upper balcony.

All the individual parts—comprising the tube, staircase, buttresses, reservoir, balcony, and spire—are monolithic, and the whole of the reinforcement is most thoroughly interconnected. The weight of the construction at the top of the lower portion of the tower—that is, at the height of 3.50 metres above the terrace—is about 100,000 kilogrammes.

It should be added that the wind pressure, according to its direction, imposes very considerable loads upon the counterforts at one side of the tower, while the opposite counterforts are practically relieved of load.

So far as concerns expansion, it should be remembered that the end of the tube which extends above the reservoir is exposed to relatively rapid temperature changes; while, on the other hand, the part protected by the water in the reservoir is maintained at a temperature which varies very little indeed.

The flue of the hot-water boiler in the basement (see Fig. 173) passes through the terrace, then through the reservoir, in company with the shaft of the upper portion of the tower, and terminates at a convenient height above the balcony formed upon the top of the reservoir.

147. Concrete Facing Slabs.—In order to obviate the necessity for covering the cement surfaces of the walls with glazed stoneware, all the exterior facings were composed of slabs, from 3 to 4 centimetres thick, moulded in advance in courses about 3.5 centimetres high, this being the most suitable height for making a good layer of concrete. These facing slabs took the place of timber moulds for depositing the inner concrete of the walls after the reinforcement had been disposed in accordance with the working drawings. The villa embodies the first practical application of concrete-steel facing slabs in the manner described, but we may mention that the same system has since been adopted by several licensees under the Hennebique system, notably in the bridges of Soissons and Decize. The method includes a new and ingenious application of the principle of transverse reinforcement in

the form of flat bands of steel, and is particularly worthy of notice because it offers a ready means of obtaining natural facings of different grain and colour without the necessity for covering the surface with paint or a veneer of coloured tiles or terra-cotta.

CHAPTER IX

A Six-Storey Factory Building, Brooklyn—Boiler-House at Creosoting Works in Texas—Foundations for a Factory in Essex

A SIX-STOREY FACTORY BUILDING, BROOKLYN

148. General Description.—The building of which details are here given was built in 1904 for the Thompson & Norris Company, at the corner of Prince Street and Concord Street, Brooklyn, in accordance with the system of the Expanded Metal and Corrugated Bar Company, of St. Louis,—a method of construction that has since been introduced into Great Britain under a different name by the Patent Indented Steel Bar Company, of Westminster.

The designs were prepared by Mr. H. C. Millar and Mr. Horace I. Moyer, who also superintended the execution of the work, the Thompson & Norris Company being the builders as well as the owners.

Fig. 179 is a perspective view of the building, which covers a ground area of 136 ft. by 80 ft., and rises to the height of 72 ft. from ground level to the top of the cornice. The roof has a slight inclination on either side of the central ridge, and is provided with five skylights. The basement covers the whole area beneath the ground floor, and vaults extend beneath the pavements in Prince Street and Concord Street.

Practically the whole weight of the structure is carried by exterior and interior columns of concrete - steel, the brick walls being 8 in. thick at the bottom and 6 in. thick at the top. All interior partitions are formed of concrete reinforced by indented bars and expanded metal. The windows have metal sashes, and are filled with armoured

glass. The beams and floor slabs are of concrete reinforced by indented bars, the form of which is illustrated by Fig. 180. Fig. 181 is a view illustrating the general arrangement

FIG. 179.—View of Factory Building, Brooklyn.

of the columns and floor system. At one corner of the factory twelve panels of the ground floor are omitted to provide for the construction of a two-storey engine-room and boiler-house, from which a circular brick chimney rises to the height of 125 ft. At the back of the building

a lift well passes through all the floors, which are also in communication by stairways. These details, as well as

FIG. 180.—Patented Indented Bar.

the ducts for warming and ventilating the building, are of concrete-steel.

FIG. 181.—Interior View, showing arrangement of Columns and Floor Beams.

149. **Basement.**—Excavation for the basement was carried to the depth of about 11 ft. Fig. 182 is a plan

which shows the extent and general construction of the basement vaults.

The retaining walls comprise vertical slabs of concrete reinforced by ½-in. indented bars carried 2 ft. below the basement floor to a concrete-steel footing, and stiffened by buttresses as represented in Fig. 183. The top of the wall is formed by longitudinal beams reinforced with indented bars, some of which are bent up near each buttress as shown in the drawing. These longitudinal beams

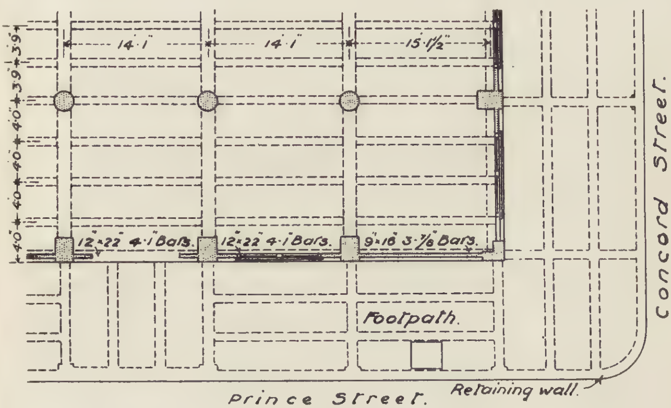


FIG. 182.—Ground Floor Plan.

support one end of the transverse beams connecting the retaining wall with the main beams of the building.

Except in one or two cases the transverse beams support two lines of intermediate longitudinal beams dividing the vault roofing into three series of panels, of which the series nearest to the building is provided with ventilating lights and the other two are filled by concrete-steel slabs. In places where no intermediate beams occur the transverse beams are spaced more closely, as indicated in Fig. 182.

150. Column Foundations.—As a general rule the footings for the columns were formed without timbering,

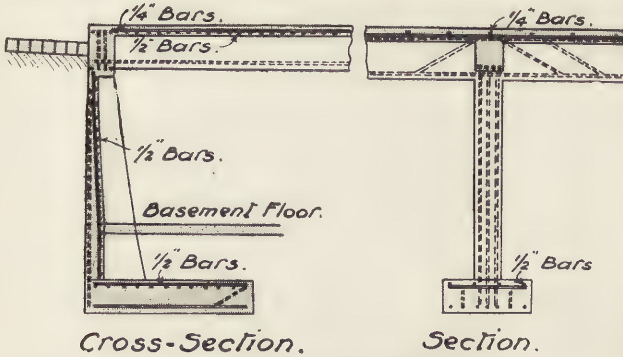


FIG. 183.—Retaining Walls.

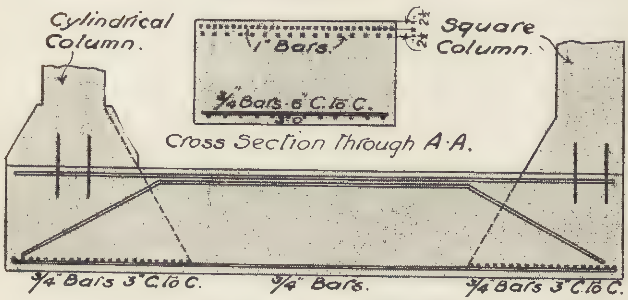


FIG. 184.—Sections of Two-Column Footing.

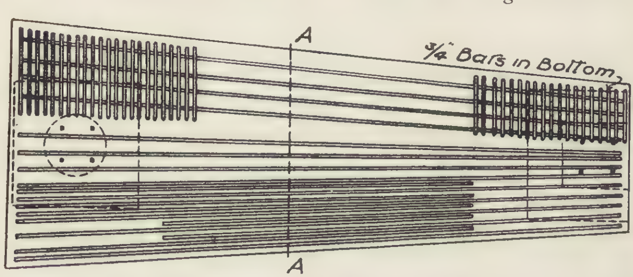


FIG. 185.—Plan of Two-Column Footing.

being moulded in pits dug from 5 ft. to 8 ft. below basement floor level.

The footings for single columns vary from 7 ft. to 8 ft. square, and are from 8 in. to 11 in. thick. They are reinforced by $\frac{3}{4}$-in. indented bars in two series, one at right angles to the other, situated 3 in. above the lower surface of the concrete, and proportioned to give a cross section of metal equal to from 0.84 sq. in. to 1.15 sq. in. per foot of width. The footings were built independent of the columns, and were provided with four projecting anchor bars, as shown in Fig. 187.

Footings designed to support two columns were necessary along the inner sides of the site, these footings having to support the columns along the party walls and the adjacent columns in the first interior row. The double footings, as shown by Fig. 184, are wide girders reinforced by 1-in. bars near the top and by $\frac{3}{4}$-in. bars near the bottom, some bars of the upper series being bent downwards near the ends, as represented in Fig. 184.

Fig. 185 is the plan of a typical double footing. The footings were built independent of the columns, with which they are connected by anchor bars as shown in the section. Multiple footings were constructed at two corners of the building,—one for the support of four columns, and the other to support two columns and the boiler chimney, a structure representing the dead load of nearly 900,000 lb.

Fig. 186 contains a plan and section of the last-mentioned footing, which has the length of 18 ft. 3 in., the mean width of 15 ft. 6 in., and the depth of 1 ft. 9 in. It is reinforced by 1-in. indented bars 2 in. below the top and $\frac{3}{4}$-in. indented bars 3 in. above the bottom disposed as illustrated in the drawings.

151. Wall Columns.—All the wall columns, except those at the four corners of the building, are of rectangular cross section, ranging from 20 in. by 26 in. at the base to 10 in. by 18 in. at the top. They are reinforced by four indented bars made up of several lengths, the sizes of which decrease from $1\frac{1}{4}$ in. at the base to $\frac{1}{2}$ in. at the top. The bars have squared ends, and the joints are made by splice bars and a binding of steel wire.

The vertical bars of the reinforcement are wired to horizontal rectangular frames spaced 12 in. apart, and made of $\frac{1}{4}$-in. indented steel bars.

At the corners of the building the wall columns are of L-shaped cross section, and are reinforced by five vertical bars, jointed and connected by lateral frames in the manner already described.

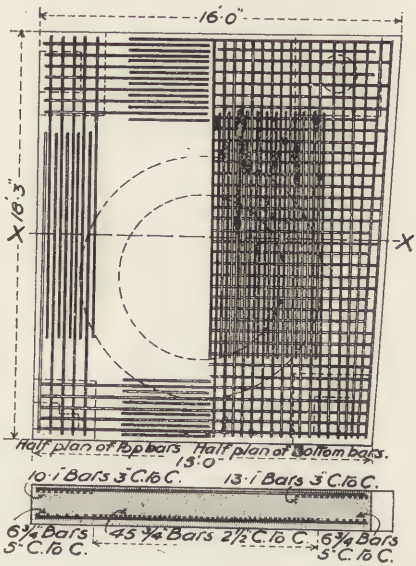


FIG. 186.—Plan and Section of Footing for Chimney and Four Columns.

152. Interior Columns.—These members are of a special type, first used in this building. Their cross section is circular, ranging from 28 in. at the base to 12 in. at the top, exclusive of plaster finish.

Fig. 187 is a drawing containing full details of the construction. The reinforcement consists of four vertical bars graduated in size from 1 in. at the base to $\frac{1}{2}$ in. at the

top, and a sheathing of No. 10 expanded metal with 3-in. mesh wired so securely at the overlapping edges that the strength of the joint was greater than that of the sheet metal.

As shown in Fig. 187, the vertical bars project to a sufficient distance above each floor level to provide for jointing the different lengths of the columns.

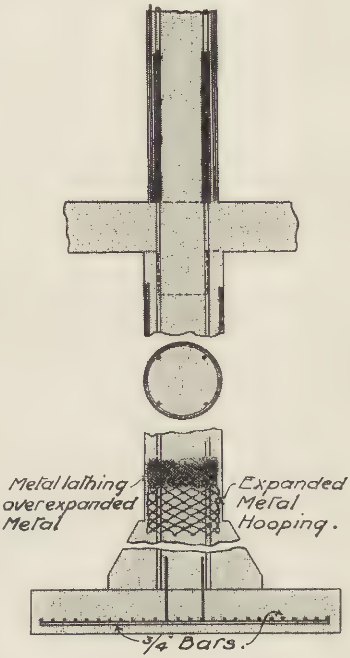


FIG. 187.—Section of Interior Column.

153. Method of Moulding Interior Columns.—The columns were moulded in cylinders of No. 10 expanded metal, to which were wired the vertical bars of the reinforcement. An outer sheathing of metal lathing with $\frac{1}{2}$-in. mesh was wrapped round each cylinder. Then the cylindrical moulds were placed in position, and secured by wiring the vertical bars to the anchor bars projecting above the footings.

Concrete was deposited in the usual manner, and the outer surface was plastered with a $\frac{5}{8}$-in. coat of 1 : 3 cement mortar trowelled smooth.

The column bases are extended as shown in Fig. 187, the batter of the sides being at the angle of 60 degrees with the horizontal.

154. Floor Constructions. — Fig. 188 contains a plan and two sections typical of the floor construction, which is practically the same in every storey, the only

differences being in the dimensions of the beams and slabs.

The main beams are from 15 ft. to 16 ft. long between the centres of the supports, and spaced from 12 ft. to 15 ft.

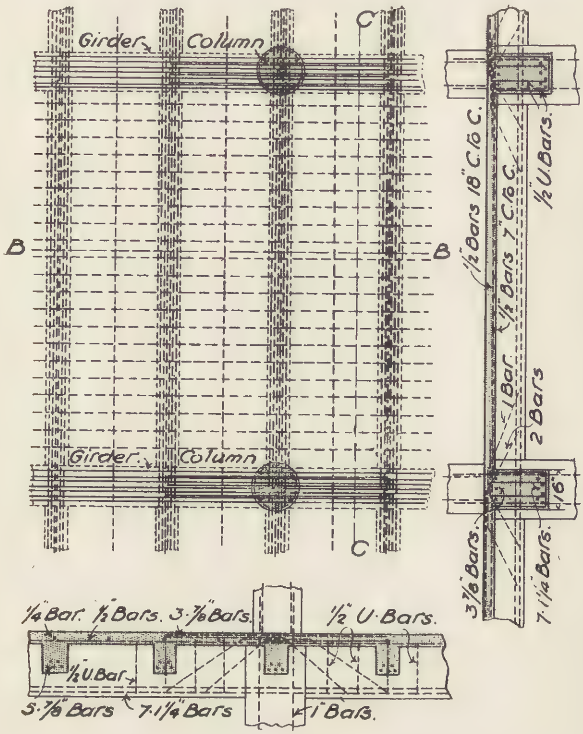


FIG. 188.—Plan and Sections of Floor Construction.

apart centre to centre. The secondary beams are about 3 ft. 9 in. apart centre to centre, and the complete system of beams on each floor is connected by a continuous slab of concrete-steel.

On the ground floor the main beams measure 16 in. wide by 26 in. deep, and are reinforced by seven $1\frac{1}{4}$-in. indented bars near the lower surface. The secondary beams are 9 in. wide by 16 in. deep, reinforced by five $\frac{7}{8}$-in. indented bars similarly placed.

On the upper storeys the main and secondary beams and floor slab are of generally similar construction, but the dimensions and reinforcement are varied in accordance with the loads to be sustained.

All the floors are finished with a granolithic surface layer $\frac{1}{2}$ in. thick.

155. Roof.—The roof system comprises concrete-steel rafters spaced 14 ft. apart, measuring 12 in. wide by 16 in. deep, and reinforced by three $\frac{3}{4}$-in. indented bars near the lower surface of the concrete. These rafters have a very slight inclination towards the ridge, and are connected by a continuous slab which performs the duty of the purlins and boarding used in ordinary roof construction.

In the upper surface of the concrete, timber battens are embedded, to which a 5-ply felt waterproofing course was nailed, and over this a tiled roof covering was laid.

156. Loads.—The interior columns were designed for maximum loads of from 106,000 lb. to 480,000 lb. each.

The calculated floor and roof loads were as follow:—

Ground floor	400 lb. per sq. ft.
Other floors	200 ,,
Roof	50 ,,

All beams were proportioned by the formula of Professor W. K. Hatt, as modified by Professor L. J. Johnson.

157. Concrete Data. — For foundations, footings, retaining walls, and pavement slabs the proportions of the concrete were 1 part Portland cement, 2 parts sand, and 5 parts trap rock crushed to pass through a $1\frac{1}{2}$-in. ring.

For columns, beams, and floor slabs the proportions of the concrete were 1 part Portland cement, 2 parts sand, and 4 parts broken stone crushed to pass through a $\frac{3}{4}$-in. ring.

All particles of rock were allowed to remain in the broken stone, with the exception of dust.

All concrete was machine-mixed, and sufficient water was used to form a very wet mixture.

BOILER-HOUSE AT CREOSOTING WORKS IN TEXAS

158. General Description of Works.—Fig. 189 is a view of the creosoting works built at Somerville, Texas, for the Atchison, Topeka, and Santa Fé Railroad Company.

The rectangular building in the illustration is the boiler-house, the building to the right hand is occupied by cylinders for the treatment of railway sleepers by creosote, and the black structure behind the boiler-house is a row of creosote tanks each supported on a framework of columns and beams.

The two buildings and the tank supports are of concrete-steel, designed in accordance with the Indented Bar system.

159. Boiler-House.— In this article we confine attention to the construction of the boiler-house, which measures 72 ft. long by 41 ft. wide, and is 18 ft. high from ground level to the roof, the latter having a slight fall from the centre to the side walls.

A ventilator 24 ft. long by about 6 ft. wide occupies a central position in the roof, and an awning with the projection of 7 ft. extends for a length of about 50 feet along the front wall.

As will be seen from the following particulars, the construction is very simple.

Figs. 190 and 191 are respectively a plan and a section of the building, which is supported by eighteen columns, seven each in the front and back walls and two each in the end walls.

160. Columns.—All the columns measure 16 in. square, being reinforced by eight $\frac{3}{4}$-in. indented bars up to the height of 9 ft. (see Fig. 192), and by four similar bars from that level to the top (see Fig. 193). The vertical bars are tied laterally by a binding of soft iron wire at intervals of 12 in. apart.

At the top of each column in the front and back walls a bracketed extension is provided on the inner side to form

a rigid connection between the columns and the roof beams, as may be seen in Fig. 191.

161. Walls.—Between the columns are concrete-steel

Fig. 189.—Creosoting Works at Somerville, Texas.

wall panels 4 in. thick, in which the reinforcement consists of vertical and horizontal ½-in. indented bars spaced 3 ft. apart near each face of the concrete, the horizontal rods passing through the columns. Typical details of the construction are illustrated in Fig. 194.

162. Roof Beams and Slab.—The roof beams are 16 in. wide, with the maximum depth of 42 in. at the centre. They are reinforced by six 1¼-in. indented bars in two rows near the lower surface of the concrete, with diagonal bars near each end to resist shearing stresses.

Each end wall of the boiler-house is capped by a beam 8 in. wide by 34 in. deep at the centre. These beams are incorporated with the end columns and walls, and are reinforced by three ¾-in. indented bars near the lower surface of the concrete.

The roof slab is 6 in. thick, reinforced by ½-in. longitudinal indented bars spaced

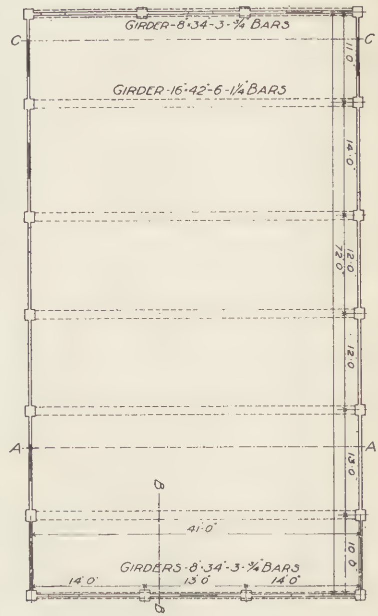


FIG. 190.—Plan of Boiler-House.

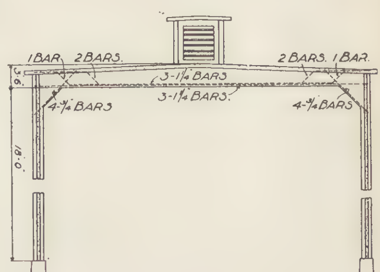


FIG. 191.—Section of Boiler-House.

5 in. apart and placed near the under side of the concrete.

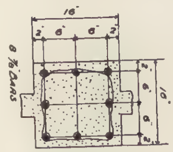


FIG. 192.

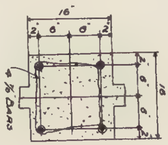


FIG. 193.

Sections of Column.

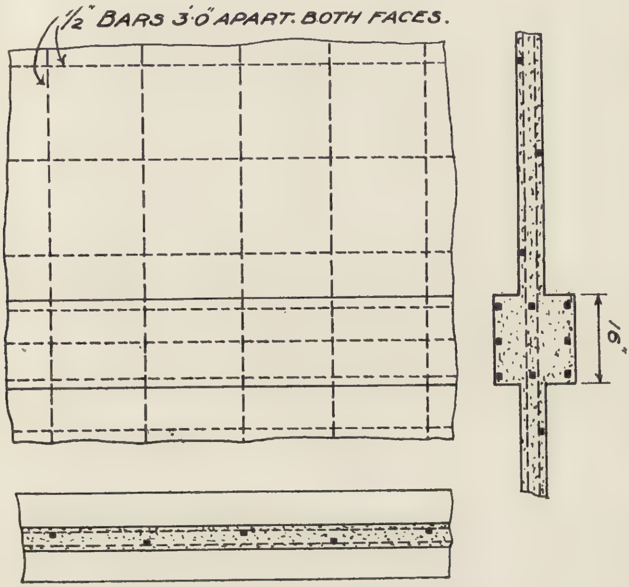


FIG. 194.—Details of Wall Construction.

Similar bars 6 ft. long are embedded over the roof beams near the top of the slab, for the purpose of withstanding

tensile stresses developed by continuous-girder action. Fig. 195 contains sections of the roof slab and beams.

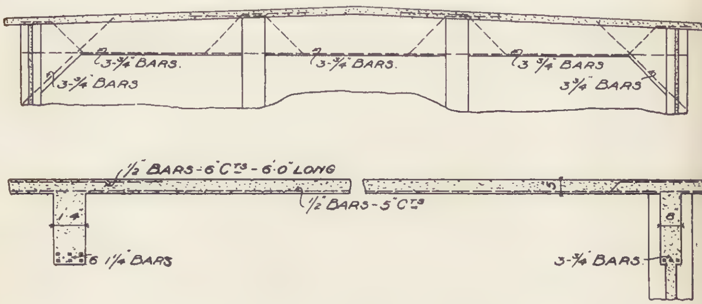


FIG. 195.—Details of Boiler-House Roof.

163. Ventilator. — The ventilator has walls of 1 : 3 cement mortar 4 in. thick, monolithic with the roof slab, and

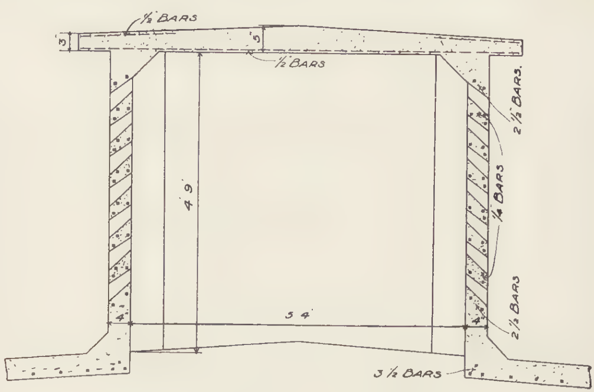


FIG. 196.—Section of Ventilator.

reinforced by $\frac{1}{4}$-in. indented bars placed horizontally (see Fig. 196). The sides of the ventilators are provided with concrete-steel louvres having bars $3\frac{1}{2}$ in. deep at the back

and $2\frac{1}{2}$ in. deep at the front, forming openings of similar dimensions. The louvre bars are reinforced in the same manner as the other portions of the side walls.

The roof of the ventilator consists of a slab with the maximum thickness of 5 in. at the centre, sloping down to 3 in. thick at the eaves, where it projects sufficiently to deliver rain-water upon the main roof slab free from the side walls. This slab is reinforced longitudinally and transversely by $\frac{1}{2}$-in. indented bars spaced 18 in. and 12 in. apart respectively.

164. Awning.—The front awning of the boiler-house

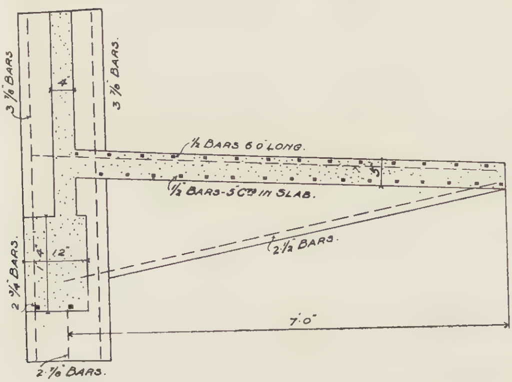


FIG. 197.—Awning in front of Boiler-House.

(see Fig. 197) is formed by a 5-in. slab of concrete-steel reinforced by $\frac{1}{2}$-in. indented bars arranged longitudinally and transversely, the spacing in each case being 5 in., and by longitudinal bars 6 ft. long over each of the supporting brackets. The latter project from the main wall columns, and are 12 in. thick by 20 in. deep, close to the wall, being reinforced by two $\frac{1}{2}$-in. indented bars parallel with the lower surface to aid the concrete in resisting compression.

165. Staircase.—Fig. 198 is a section illustrating the construction of a stairway in the pump room. The concrete steps are reinforced by $\frac{1}{2}$-in. indented bars spaced 3 in.

apart, centre to centre; the minimum thickness of concrete being 4 in., measured in a direction perpendicular to the surface of the slope below the stairs.

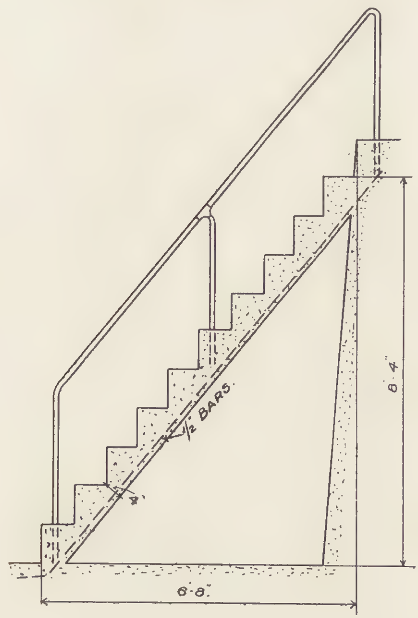


FIG. 198.—Stairs in Boiler-House.

FOUNDATIONS FOR A FACTORY IN ESSEX

166. General Description.—The foundation works here described were executed for supporting the brick superstructure of the new premises built for Messrs. J. C. & J. Field Ltd., at Rainham, Essex. The architects for the works were Messrs. Scott, Hanson, & Fraser, of Basingham Street, London, E.C., and the contractors were Messrs. W. King & Sons, of London. Fig. 199 is a view of the building taken during construction.

The concrete-steel construction, designed in accordance with the Coignet system, comprises three principal items—(1) continuous foundations for the boiler-house 92 ft. 6 in. long by 72 ft. 6 in. wide; (2) foundations for three Galloway boilers; and (3) continuous foundations for a building 271 ft. long by 41 ft. 6 in. wide.

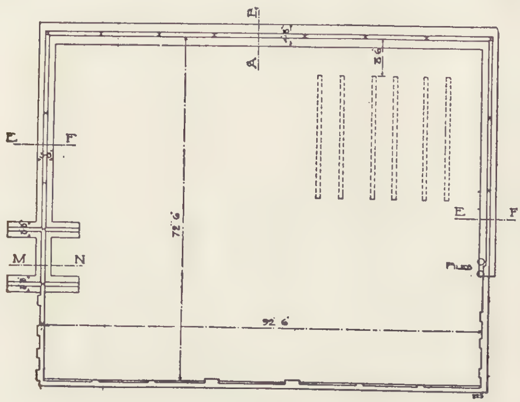
Owing to the treacherous nature of the ground, which is composed of hard silt to the depth of 3 ft., resting upon a bed of peat about 25 ft. thick, it was impossible to build

FIG. 199.—View of Factory under Construction.

with safety upon ordinary foundations. The use of timber piling would have been expensive, and might have been prejudicial to the permanence of the superstructure, on account of the possible decay of the timber in consequence of the waterlogged condition of the soil. Having considered the problem in its various aspects, the architects decided to employ continuous foundation slabs of concrete-steel, by which the loads are distributed over so large an area as to keep the unit pressure within the safe bearing power of the ground.

167. Boiler-House Foundations.—The general

nature of the foundations for the boiler-house will be gathered by reference to Fig. 200. The concrete-steel foundation extends around three walls of the building, and consists of footings 4 ft. 8 in. wide by 3 in. thick along two walls, and $4\frac{3}{4}$ in. thick along the third wall, distributing the weight of the building at the rate of 3 cwt. per square foot over the surface of the ground. Upon this footing a con-



PLAN

FIG. 200.—Plan of Boiler-House.

tinuous beam 10 in. wide by 12 in. deep was formed for the purpose of supporting the brick walls of the boiler-house, in which it was originally proposed that steel stanchions should be built for the purpose of supporting the roof principals, and bolted at the foot to the concrete-steel beam. These stanchions have been replaced by concrete-steel columns and lintels resting directly upon the foundation beams of the building. As these members are built monolithic with the foundations, the framework is far more rigid, and at the same time less expensive than the hybrid construction at first contemplated.

At one place where the weight transmitted by the stanchion amounts to nearly 18 tons the foundation beam has been supplemented by two transverse projections (see

MN, Fig. 1), and at the opposite side of the building, where the load reaches nearly 36 tons, two piles have been driven.

Details of the concrete-steel construction will be found in Fig. 201, which contains sections through the lines AB, EF, and MN, in Fig. 200.

168. Boiler Foundations. — The position of these foundations is shown in Fig. 200. Fig. 202 is a plan of the foundations to a larger scale, and a section of

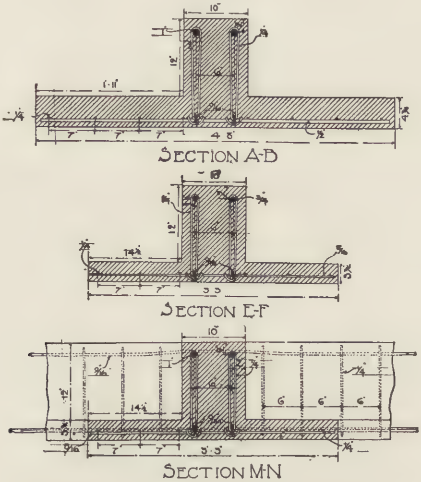


FIG. 201.—Details of Slab and Beams.

the slab and beams will be found in Fig. 203. The weight of the boilers is spread over an area of about 40 ft. square, thus reducing the load to 3 cwt. per sq. ft. Every possible care was taken to avoid disturbance of the upper stratum of silt. A layer of broken brick a few inches thick was spread over the surface of the ground, and when this had been well levelled a bed of concrete 5 in. thick was deposited, and reinforced by a framework of bars, as shown in Fig. 203, one drawing being a section through the line CD in Fig. 202, and the other a part section through EF in Fig. 203.

Upon the general foundation slab six parallel concrete-steel beams were built, as represented in Fig. 202, to

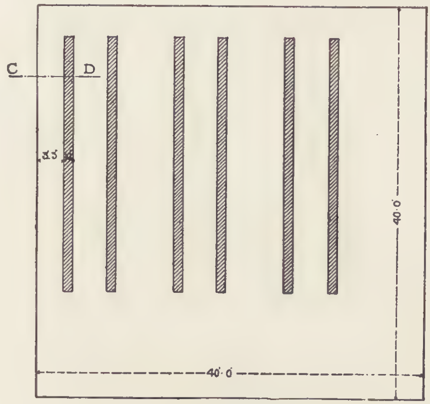


FIG. 202.—Plan of Boiler Foundations.

support the fireclay seating blocks for the boilers. It should be observed that, owing to the great rigidity and elastic

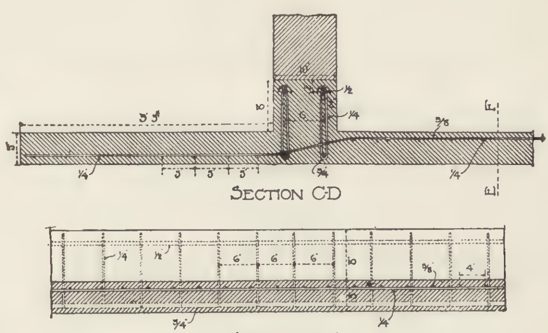


FIG. 203.—Details of Slab and Beams.

strength of the reinforced construction, it was only necessary to make the slab a few inches thick. If plain concrete had

been employed a thickness of several feet would have been demanded, and even then cracks would certainly have appeared, and the weight to be spread over the surface of the ground would have been so much increased as to render the employment of the material impracticable. Fig. 204 is a view of the completed foundations with the boilers in position.

169. Shed Foundations.—Fig. 205 is a plan of the shed, 271 ft. long by 41 ft. 6 in. wide, for which the

FIG. 204.—Completed Foundations.

foundations were of generally similar character to those of the boiler-house, except that the slab is in the form of a hollow rectangle beneath all four walls of the building. As shown in Fig. 206, the slab is 4 in. thick, and upon it is moulded a continuous beam 10 in. wide by 12 in. deep, the details of the reinforcement being as represented in the two sections.

Fig. 207 is a view of the foundation before removal of the moulds. As in the case of the boiler-house, concrete-steel columns have been substituted for the steel stanchions included in the original scheme. The weight on each

column is about 9 tons, the columns being spaced 12 ft. apart centre to centre.

170. Construction.—It is stated that the concrete-steel work was carried out altogether by unskilled labour, under

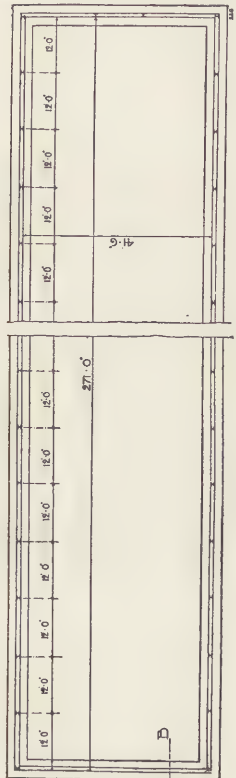


FIG. 205.—Plan of Shed.

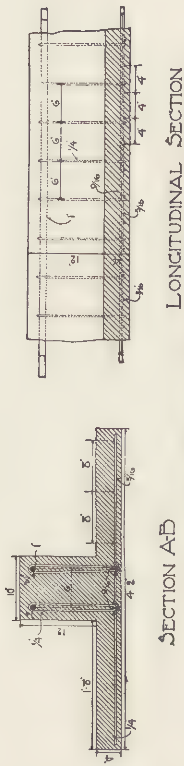


FIG. 206.—Details of Slab and Beams.

the superintendence of a clerk of works and contractors' foreman, neither of whom had any previous experience of the material. We presume, however, that all necessary supervision was also exercised by the representatives of the

patentees, for it is well known that the execution of work in reinforced concrete is an operation demanding above all

FIG. 207.—View showing Construction of Foundation.

things expert superintendence and the most careful attention to various details connected with the preparation and deposition of the concrete and the disposition of the reinforcement.

CHAPTER X

A FACTORY BUILDING AT YORK—COAL BUNKERS, PARIS—SCREEN FOR COAL BUNKERS, RAINHAM—FLOUR MILL AND GRANARY, SWANSEA—EXPANDED METAL SILOS

A FACTORY BUILDING AT YORK

171. General Description.—This building constitutes an important addition to the extensive works of Messrs. Rowntree & Co. at York. It furnishes an excellent example of framed construction in reinforced concrete, in some measure akin to the steel-frame construction which has been adopted so extensively in the United States.

Every detail of the structure from foundations to roof is built in Hennebique ferro-concrete, from the drawings prepared by Mr. L. G. Mouchel, M.Soc.C.E. (France), in accordance with the designs of Mr. W. H. Brown, architect to Messrs. Rowntree & Co. The contractors were the Yorkshire Hennebique Contracting Co., of Leeds.

The building is termed the new Melangeur block, because it has been designed specially for the operation of the *melangeurs*, or mixing machines, employed in the manufacture of cocoa and chocolate. It covers an area of 105 ft. by 76 ft. 3 in., and, as shown in Fig. 208, includes five well-lighted floors having an aggregate area of nearly 40,000 square ft. The roof is flat, and surrounded by a deep parapet which renders it available for the storage of packing cases or non-perishable goods. The total height from foundation to parapet is 93 ft. 6 in., and the heights of the different storeys from ceiling to ceiling are as follow:—

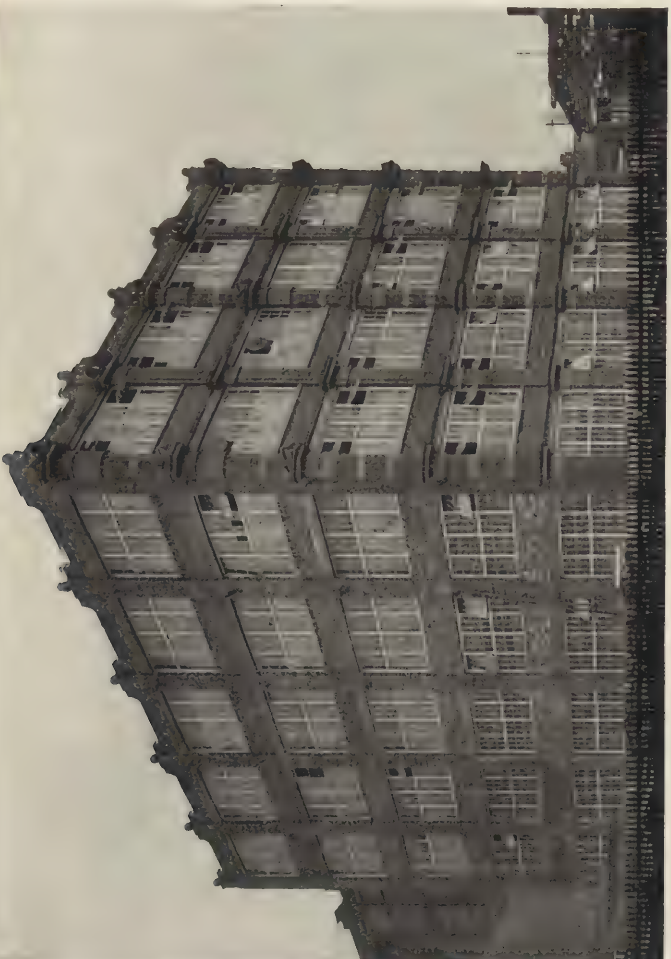


FIG. 208.—Elevation of Factory Building at York.

	Ft.	In.
Basement	8	6
Ground floor	16	7½
First floor	14	7½
Second floor	14	7½
Third floor	15	4
Fourth floor	14	6

The entrance to the building is situated at one side, access being given at ground level from other parts of the works by a covered corridor 6 ft. wide inside, with a lean-to roof, the top of which extends nearly up to the level of the first floor. The outer wall of the corridor is 9 in. thick, and is carried on a footing 8 in. thick by 20 in. wide.

Opposite the entrance door a staircase extends from the basement to the roof, and at the front of the building a lift opening 7 ft. square provides for the instalment of a lift to give facilities for the transport of materials from one floor to another.

Provision is made for fixing several lines of shafting for driving the mixing machinery, which is installed on the floor at ground level, and a platform is formed at the height of 8 ft. above floor level for the convenience of the attendants engaged in charging and discharging the melangeurs.

These are the main features of the building, the arrangement of which is further explained by the plan and section reproduced in Figs. 209 and 210.

172. Foundations.—As the soil is of somewhat compressible character the whole building is founded upon a concrete-steel slab 12 in. thick, projecting about 10 ft. in every direction beyond the outer walls, thereby distributing the load over an area of nearly 9,700 sq. ft.

The slab is situated at a depth of 12 ft. below ground level, and receives all loads from the superstructure through the bases of the columns, except the comparatively insignificant dead and live loads of the basement floor and walls. The disposition of the foundation slab and footings is clearly shown by Fig. 210.

This slab is strongly reinforced by round steel bars of suitable diameter, laid both longitudinally and transversely

near the top and bottom surfaces of the concrete, so as to enable the construction to withstand tensile stresses wherever such may be developed.

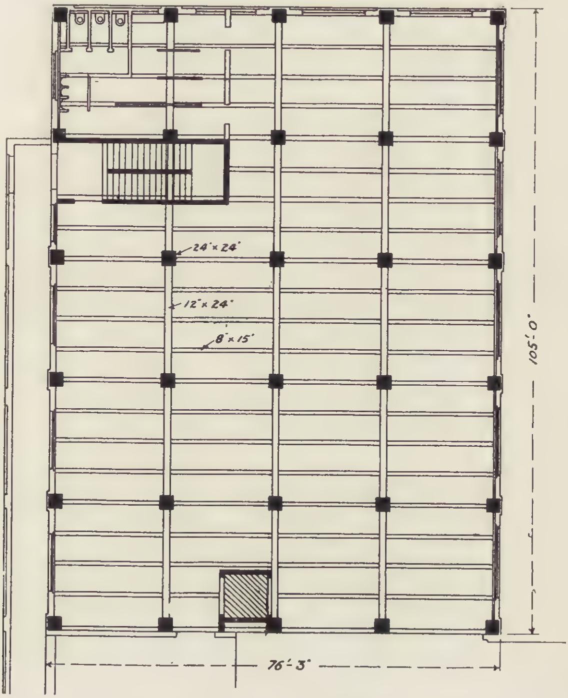


FIG. 209.—Plan of Building.

173. Columns.—In Fig. 210 typical column bases are shown in elevation. In the outer wall columns the bases measure 12 ft. square, those of the interior columns being 15 ft. square, and all the bases have the uniform height of 2 ft. 6 in. above the foundation slab.

The column bases are reinforced by a network of round

steel bars near the lower surface for resisting tension, and by vertical stirrups of strip steel for resisting shear. At the upper portion of each base the bars forming the vertical reinforcement of the column above project into the base, so that metallic connection is established between the two members in addition to the continuity of the concrete,

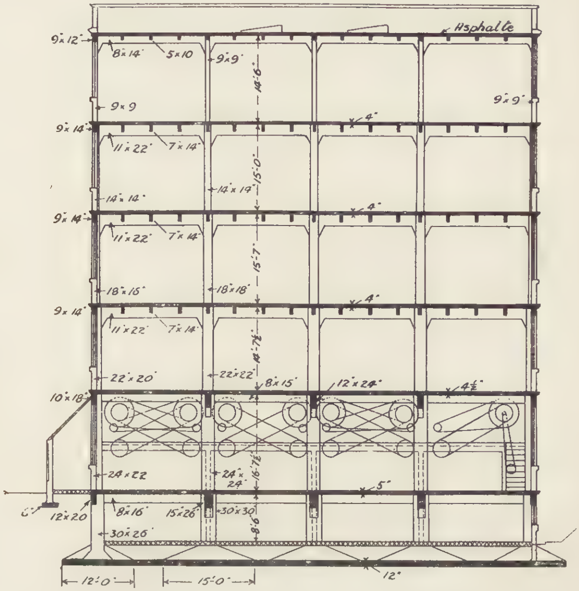


FIG. 210.—Transverse Section of Building.

similar connections being made between the column bases and the foundation slab beneath.

All the columns in the building are of uniform dimensions, although the cross sectional area varies from storey to storey as stated below. As shown in Fig. 209 they are disposed in five longitudinal rows of six each, eighteen of the columns being in the four outer walls and twelve in the interior of the building.

The dimensions are as follow :—

Basement	30 in. square
Ground floor	24 ,,
First floor	22 ,,
Second floor	18 ,,
Third floor	14 ,,
Fourth floor	9 ,,

The reinforcement of the columns comprises longitudinal round bars of steel connected at intervals by horizontal

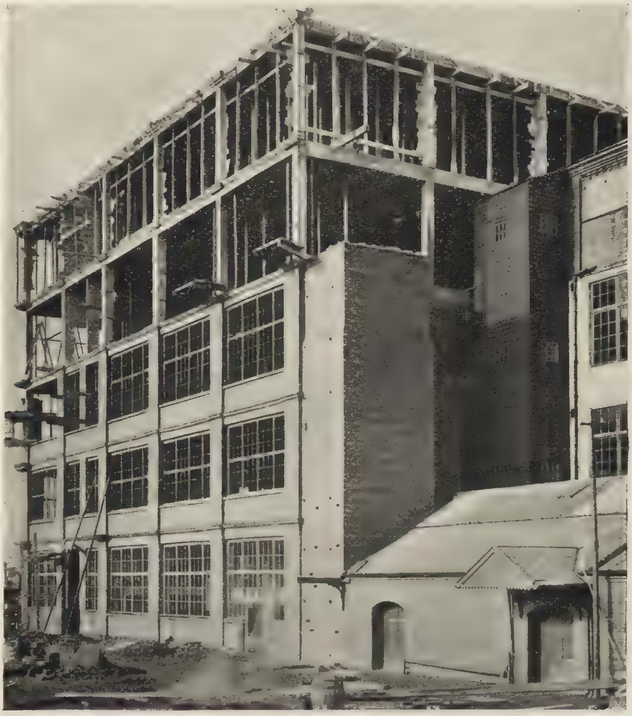


FIG. 211.—View showing Wall Columns and Beams.

links for the purpose of keeping the longitudinals in position during the concreting, and of enabling them and the concrete more effectively to resist axial and non-axial loads.

174. Main and Secondary Beams.—Figs. 211, 212, and 213 are views taken during erection by which an excellent idea of the framed design may be obtained, and which illustrate the connection of the columns in the side and end walls by concrete-steel beams. These beams are of rectangular cross section, and their dimensions are as follow:—

Ground floor . .	12 in. wide	by	20 in. deep.
First floor . .	10	,,	18	,,
Second floor . .	9	,,	14	,,
Third floor . .	9	,,	14	,,
Fourth floor . .	9	,,	14	,,
Roof . . .	9	,,	12	,,

The interior and wall columns are conncted at the floors by transverse main beams of concrete-steel with the following dimensions:—

Ground floor . .	15 in. wide	by	26 in. deep.
First floor . .	12	,,	24	,,
Second floor . .	11	,,	22	,,
Third floor . .	11	,,	22	,,
Fourth floor . .	10	,,	22	,,
Roof . . .	10	,,	14	,,

The columns and main beams are connected longitudinally by concrete-steel secondary beams, spaced from 5 ft. to 5 ft. 6 in. apart on the different floors, of the dimensions stated below:—

Ground floor . .	8 in. wide	by	16 in. deep.
First floor . . .	8	,,	15	,,
Second floor . .	7	,,	14	,,
Third floor . .	7	,,	14	,,
Fourth floor . .	5	,,	10	,,
Roof. . . .	5	,,	10	,,

Thus the main and secondary beams divide up each

floor system and the roof into a series of rectangular panels about 18 ft. by 5 ft. centre to centre, as indicated by the plan and transverse section, Figs. 209 and 210.

175. Floors and Floor Loads.—The basement floor

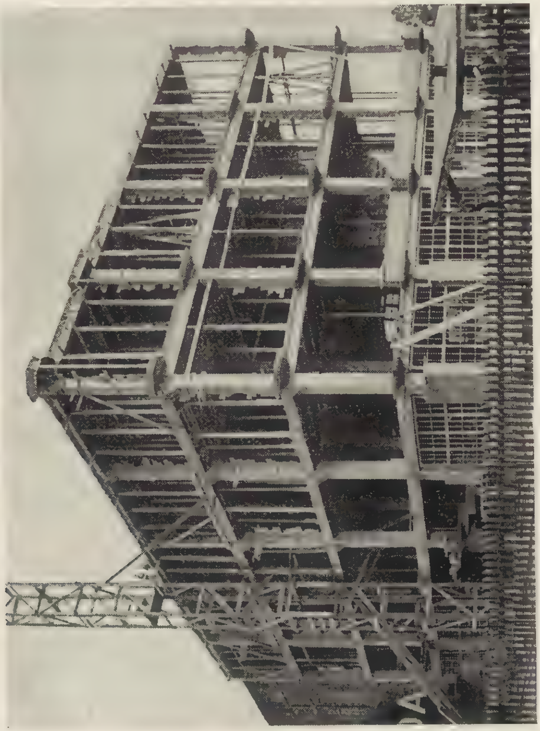


FIG. 212.—View showing Framed Construction.

slab is 9 in. thick, formed of plain concrete deposited on earth spread over the foundation slab and thoroughly rammed. Reinforcement was unnecessary here, because the floor is sufficiently supported by the solid filling beneath, and forms no part of the building regarded purely as a framed structure.

All the other floor slabs, as well as that of the flat roof, are fully reinforced.

Owing to the weight of the machinery with which the building is equipped and the considerable quantities of raw material and finished products to be handled during the process of manufacture, the various floors were designed for the following superloads:—

	Lb. per Square Ft.
Ground floor	840
First floor	456
Second floor	336
Third floor	336
Fourth floor	336

In spite of these exceptionally heavy loads the floor slabs are of very moderate thickness, ranging from 5 in. in the ground floor, 4½ in. in the first floor, to 4 in. in the second, third, and fourth floors.

As indicated in Fig. 210, none of the spans between the main and secondary beams is longer than 18 ft. or wider than 5 ft., and as the slab of each floor extends over the entire area of nearly 8,000 sq. ft. it acts in a manner analogous to a continuous beam, and is reinforced accordingly.

The principle underlying this type of design sufficiently accounts for the comparative lightness of the construction, and the fact that the entire slab of each floor virtually constitutes a compression flange common to all the supporting beams accounts in a large measure for its great strength and rigidity.

The flat roof resembles the floors in general design, but as the load to be carried is comparatively small the slab is only 3½ in. thick. Although the concrete is of such quality as to be capable of resisting the percolation of water, it has been thought desirable as an additional safeguard to cover the upper surface with a layer of asphalt.

176. Walls.—Owing to the fact that the floor and roof loads of the entire building, and the dead weight of the structure itself, are transmitted from member to member until they reach the main columns, and are

transmitted thence directly to the foundations, the exterior walls have no duties to perform beyond those of affording

FIG. 213.—View showing Construction of Wall Columns.

shelter from the weather and of contributing to the convenience of interior accommodation.

The walls of a building designed in the manner exemplified by this block can be as thin or as thick as may be

thought desirable, and the actual thickness adopted is governed by the nature of the work to be conducted rather than by structural considerations. In this particular example it has been thought desirable to make the walls 12 in. thick, but from a structural standpoint they could

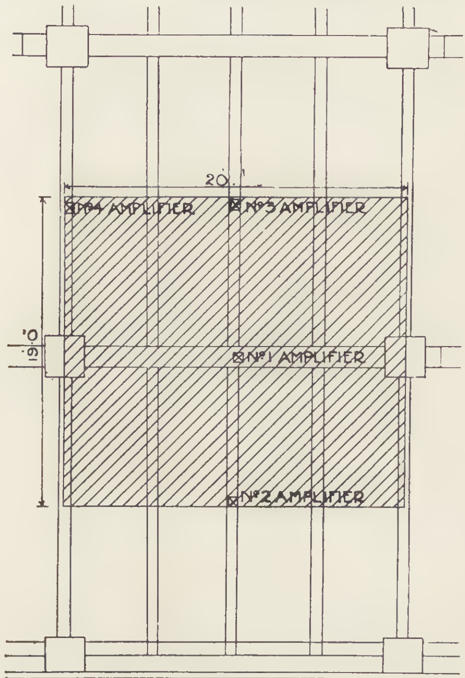


FIG. 214.—Plan of Floor Panel Tested.

just as well have been 6 in. or even 2 in. thick, or omitted altogether, as in the case of the north front of the Transit Sheds at Manchester Docks (see Article 13).

As illustrated in Fig. 210, the walls are of uniform thickness from top to bottom of the building, and are continued for a height of 4 ft. 9 in. above the surface of

the roof, thus forming a parapet permitting the top of the fourth storey to be used, if occasion should demand, as an additional floor.

FIG. 215.—View of Test Load on Ground Floor.

177. Floor Tests.—In September 1906 two tests were made upon a panel of the ground floor before the installation of the machinery.

The area tested is shown in Fig. 214, where the positions of the amplifying deflectometers are indicated.

In Test No. 1, 44,650 bricks, weighing 7.4 lb. each, were stacked upon the panel, constituting a total weight of 330,000 lb., equal to the specified superload of 840 lb. per sq. ft.

In Test No. 2 the load was increased by the weight of 22,325 bricks, making the total load of 495,000 lb., equal to 1,260 lb. per sq. ft., or 50 per cent. more than the specified superload. Fig. 215 is reproduced from a photograph of the bricks representing the maximum test load.

Sixty per cent. of the load was then taken off, and finally the remainder was removed. The subjoined table gives the deflection in millimetres, measured before commencing, during and after testing. It should be noted that on removal of the load all the beams assumed their original form, thus demonstrating the perfect elasticity of the construction.

RESULTS OF FLOOR TESTS AT YORK.

No. of Deflectometer.	Clear Span of Beam.	Deflection in Millimetres.				
		Load per Square Foot.				
		Nil.	840 lb.	1,260 lb.	504 lb.	Nil.
	Ft. In.					
1	18 3	0.00	2.10	3.30	0.80	0.00
2	16 6	0.00	2.00	2.40	1.60	0.00
3	17 6	0.00	2.40	3.30	1.20	0.00
4	15 6	0.00	0.30	0.50	0.30	0.00

COAL BUNKERS, PARIS

178. General Description.—The building here described was built by M. Edmond Coignet at the generating

Fig. 216.—Coal Bunkers under Construction at the Champs Elysées Generating Station.

station from which electricity is supplied to the Champs Elysées district of the French capital. The works in question are situated on the bank of the Seine at Levallois-Perret, a suburb which lies outside the fortifications on the north-west of the city.

The eight bunkers, each with a storage capacity of some 240 cubic metres, form one building 50 metres long by 47 metres wide, and 12.50 metres high to the eaves.

This building was erected against the open end of the boiler-house, so that coal could be delivered directly into the stoking space in front of the boiler furnaces. Each bunker has an opening in the lower part of the wall facing the boilers, and the coal is allowed to pour through this, forming a heap the inclination of which is governed by the angle of repose of the material.

At the height of 10 metres above ground level, or 2.50 metres below the eaves, there is a floor with an area of about 200 square metres covering the whole range of silos, above each of which a rectangular opening is provided. Coal is raised to the level of the floor by a continuous bucket elevator, and carried along the building by a belt-conveyor fitted with apparatus by which the coal is automatically discharged through the eight openings in the upper floor. The upper storey of the building is of very light construction, as its only purpose is to protect the conveyor mechanism from the weather. Fig. 216 is a view of the building during construction.

179. Columns.—Fig. 217 gives details of the column construction. At the front of the building, columns of 40 centimetres square between each silo, and one at each end of the range, are carried up from the ground level to the upper floor, while at the back the longitudinal and transverse walls are entirely self-supporting. The reason for the columns in the front of the building is that the resistance of the walls is seriously diminished by the openings for the discharge of coal into the boiler-house.

180. Foundation Beams.—The walls are built upon foundation beams 38 centimetres wide by 21 centimetres deep, supported by concrete footings 80 centimetres wide by 40 centimetres deep (see Fig. 217). The longitudinal

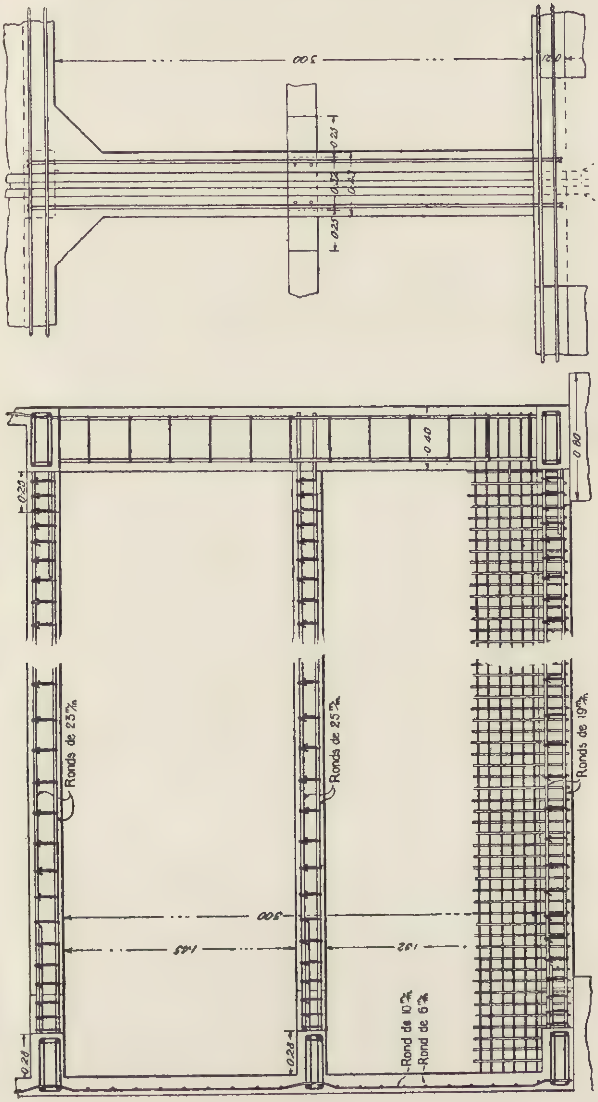


FIG. 217.—Sections of Coal Bunkers, Paris.

and transverse reinforcement of the foundation beams really forms a series of eight rectangular frames of great strength, all being connected together in such manner that they constitute a single structure strongly but not rigidly connected, the quality of rigidity being contributed by the concrete surrounding the steel network.

181. Walls.—The front and back walls of the building have the uniform thickness of 10 centimetres, the transverse walls being 8 centimetres thick. The walls are stiffened by horizontal beams projecting as ribs, 28 centimetres wide by 20 centimetres deep, on the inside of each silo (see Fig. 217). These beams are spaced at vertical distances of about 1.40 metres apart, and are certainly well adapted to the purpose of strengthening the wall construction.

In the case of the transverse walls two sets of reinforcement are employed, one for resisting tension on the inner side of the wall panels, and the other for resisting tension on the outer sides. In the front and back walls, and the end walls of the range, only one set of reinforcement is necessary, because pressure is exerted upon these walls only in an outward direction.

The horizontal beams have double reinforcement, so that they may be capable of withstanding stress induced by inward or outward pressure. Further, for the sake of ensuring ample rigidity to the whole construction the duplication of the reinforcement is extended to all the wall beams, whether subject to alternations of stress or not.

In the construction of the wall panels between the vertical and horizontal members the reinforcement takes the form of a close network of steel rods embedded in the concrete, as partly shown in the left hand drawing of Fig. 217.

SCREEN FOR COAL BUNKERS, RAINHAM.

182. General Description.—Figs. 218 and 219 contain details of the coal bunker screen, 93 ft. 6 in. long, built in accordance with the Coignet system at the new

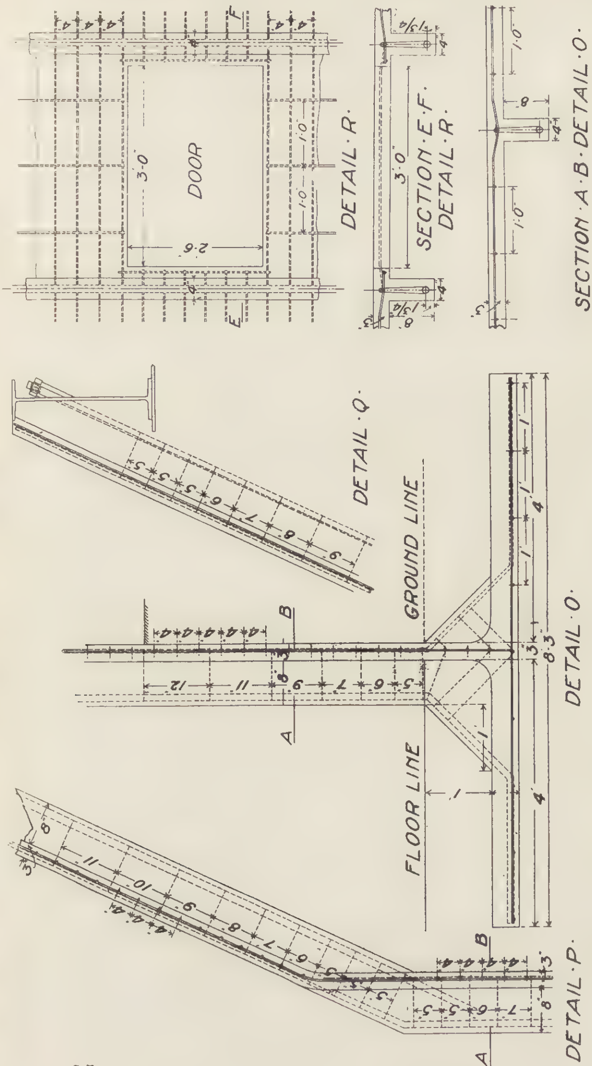


FIG. 218.—Details of Screen for Coal Bunkers.

works of Messrs. J. C. & J. Field at Rainham. The bunkers are situated in the boiler-house, shown in Fig. 200, p. 234, and extend from end to end of the building between the outer wall and the boilers.

Fig. 219 is a section illustrating the general construction of the concrete-steel screen, the principal reinforcing bars of which are bolted to the web of a rolled steel girder carried along the upper edge of the bunker.

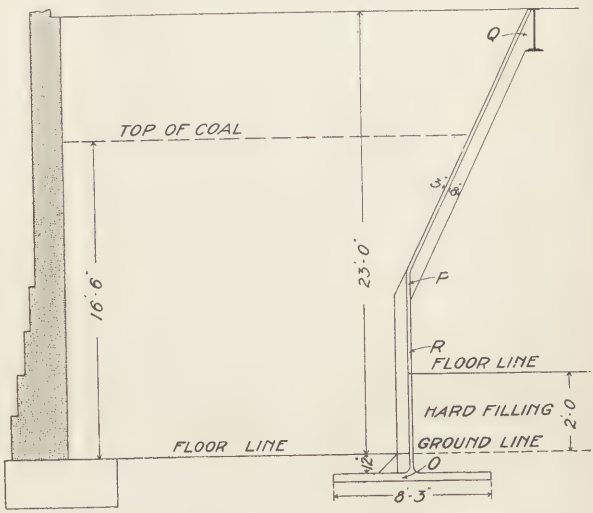


FIG. 219.—Section of Coal Bunkers, Rainham.

In Fig. 218 details O, P, and Q and section AB give particulars of the ribs and intervening panels ; and detail R and section EF show the construction of the openings through which coal is delivered to the boiler-house.

FLOUR MILL AND GRANARY, SWANSEA

183. **General Particulars.**—Fig. 220 is a photograph of the flour mill and granary built in Swansea for Messrs.

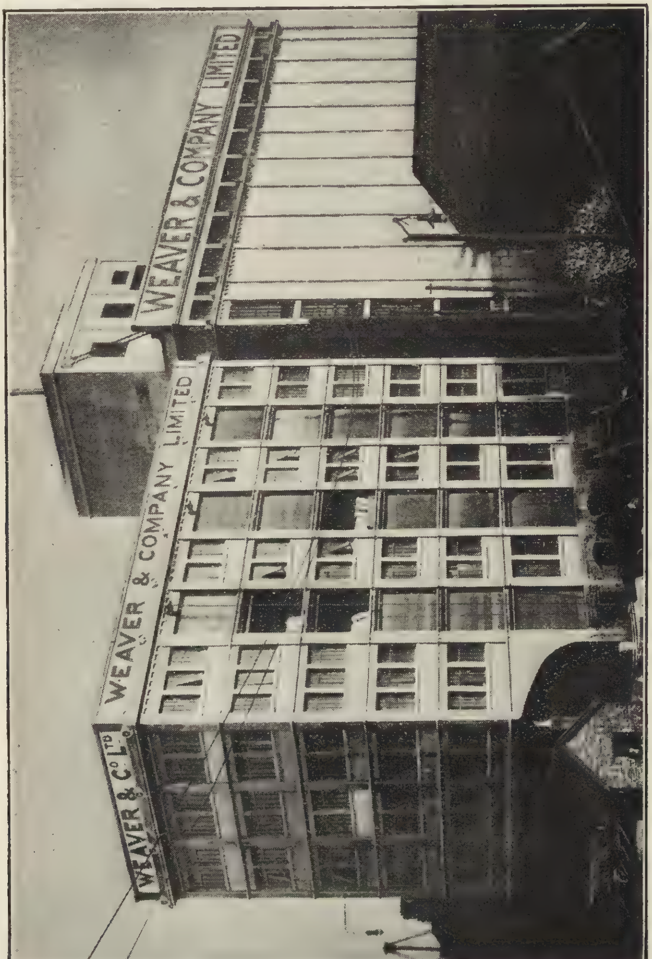


FIG. 220.—Flour Mill and Granary at Swansea.

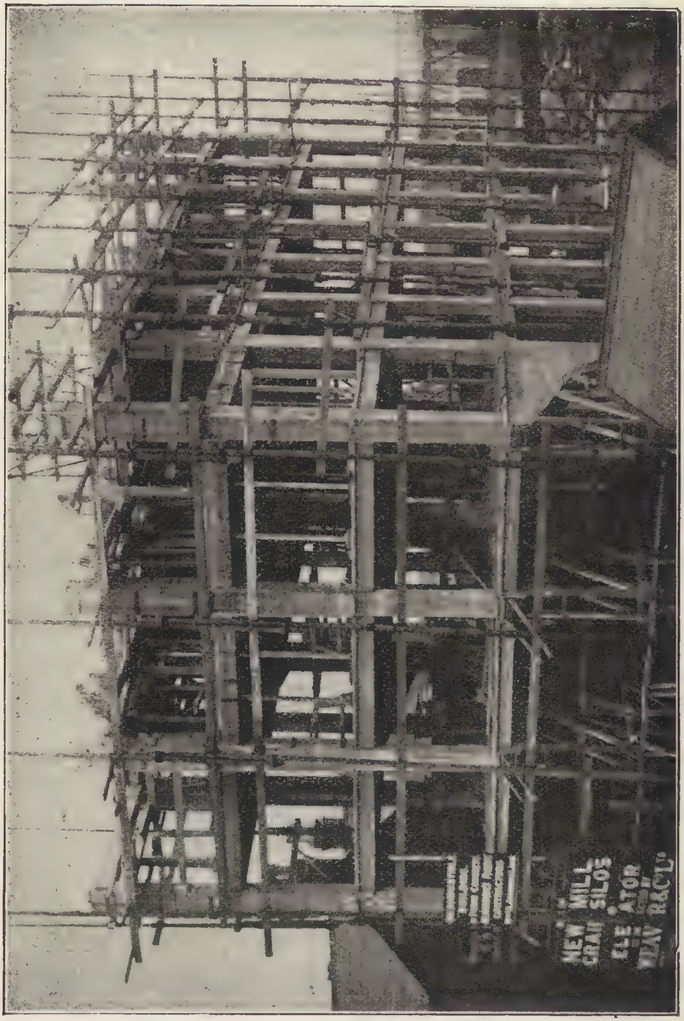


FIG. 221.—View of Mill under Construction.

Weaver & Co., from the designs of Mr. H. C. Portsmouth, M.S.A. As the foundations rest upon a layer of sand brought as ballast for ships and deposited on the soft mud forming the bed of the Swansea River, a general slab of concrete-steel was established to provide for the stability of the buildings, which are constructed throughout in accordance with the Hennebique system.

184. Description of Mill.—The flour mill is 80 ft. long by 40 ft. wide by 112 ft. high measured from foundation level to the top of the roof, which has solid parapet walls converting it into a water tank with the storage capacity of about 20,000 gallons.

As shown by Fig. 221, the mill is designed on the cage system, in which all loads are transmitted to columns and thence distributed over the foundations. Hence it was possible to make the wall panels of the minimum thickness permitted by local building regulations. The floor slabs are only $3\frac{1}{2}$ in. thick, but are so adequately reinforced and so amply supported by the main and secondary beams that they are capable of carrying a superload of 3 cwt. per sq. ft.

The grinding mills and gearing are situated upon a portion of one floor, where they involve a superload varying according to position from $3\frac{1}{2}$ cwt. to nearly 11 cwt. per sq. ft.

By Figs. 220 and 221 it will be noticed that one end of the building, representing a load of 670 tons, is carried by cantilevers, which project 14 ft. from the supports.

185. Tests. — After the building had been finished several of the floors were loaded with sacks of grain right up to the ceiling without causing appreciable deflection. Delicate measuring instruments placed beneath one panel of the floor supporting the grinding machinery registered the maximum deflection of only $\frac{3}{64}$ in., or $\frac{1}{3072}$ of the span.

186. Description of Granary. — Fig. 222 contains longitudinal and cross sections of this building, the main dimensions of which are—Length, 127 ft.; width, 48 ft. 6 in.; height above foundation level, 90 ft. 8 in. At one end, as shown in the longitudinal section, a tower projects above the general level of the roof to the height of 101 ft.

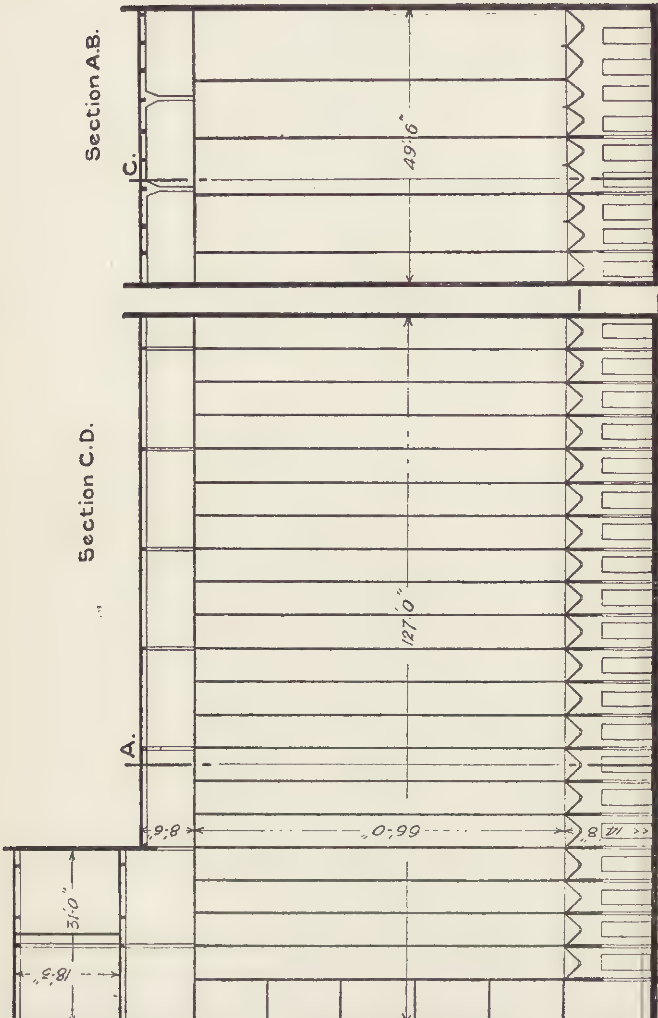


FIG. 222.—Sections of Granary.

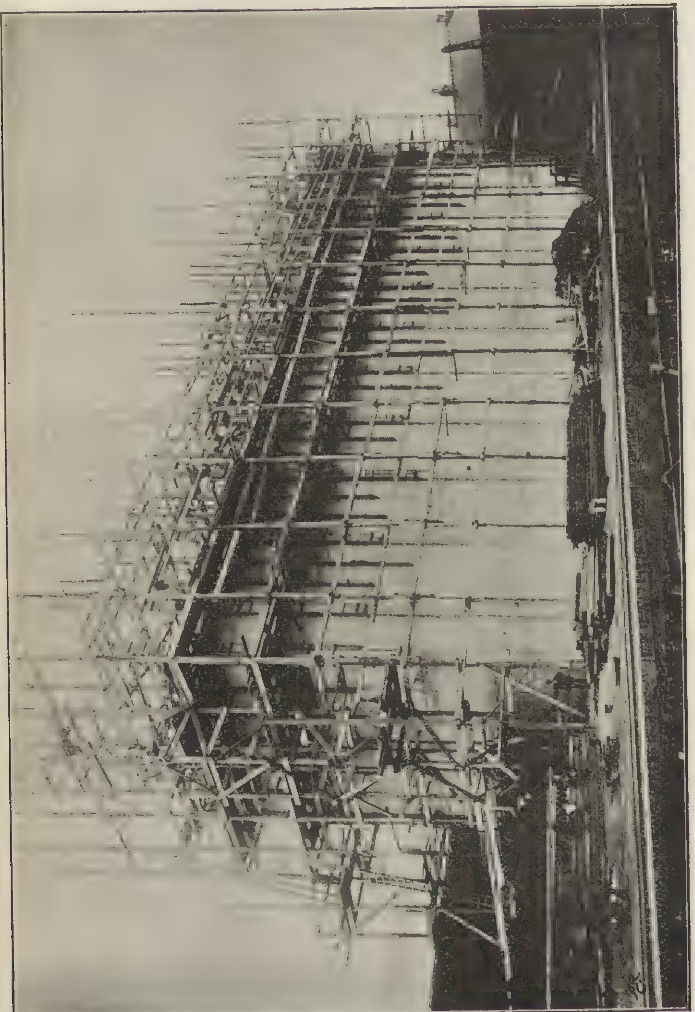


FIG. 223.—View of Granary under Construction.

8 in. above ground level. This is used for cleaning grain. The granary contains one hundred separate compartments, or silos, each provided with a hopper at the bottom for the discharge of grain, the average storage capacity of each compartment being about 70 tons, giving a total capacity of 7,000 tons.

The thickness of the external walls is 12 in. at the bottom and 4 in. at the top; the interior divisions forming the separate compartments are $5\frac{1}{2}$ in. thick up to the level of the hoppers and 3 in. thick above. A view of the building during construction will be found in Fig. 223.

187. Test.—After completion of the work a test was made, in the presence of the architect, by gauging one compartment when all the surrounding compartments were empty. The latter were then filled with grain, and on the dimensions of the centre compartment being again taken, at three points, 17 ft., 34 ft., and 61 ft. respectively, from the top, no deviation from the first dimensions was observed.

EXPANDED METAL SILOS

188. Method of Construction.—In the construction of storage bins intended for the reception of coal, grain, or other materials, expanded metal can be used with considerable advantage. The absolute connection of the strands forming the mesh ensures continuity of the reinforcement, and the comparatively small dimensions of the mesh give a satisfactory guarantee that resistance to tension will be amply provided in every part of the concrete.

The walls of the storage compartments are built of concrete having continuous sheets of expanded metal embedded near each surface so as to provide for withstanding tensile stresses, whether the adjoining compartments be empty or full, and the two layers of reinforcement are connected by ties formed of $\frac{1}{4}$-in diameter steel rods spaced 1 ft. 6 in. apart vertically and horizontally, the ends of the rods being bent over to form hooks, which are passed through the meshes of the metal. Figs. 224 and 225 illustrate the system of construction followed by the New Expanded Metal Co., of Westminster.

As shown in the drawings, ordinary rolled steel sections are used only in the construction of the hopper and the outlet. Apart from the T-bars forming the angular framework, the hopper is built of concrete with expanded metal near the outer surface, only one layer being required in this case, owing to the fact that the pressure is always exerted in an outward direction.

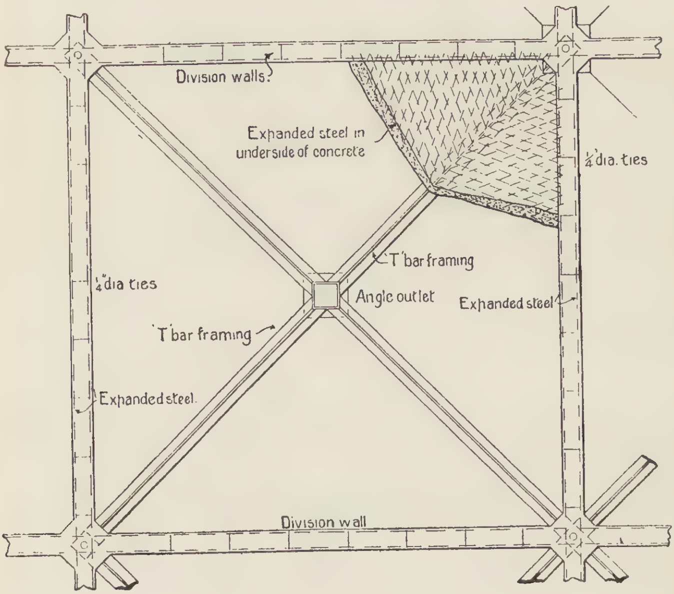


FIG. 224.

To ensure the rigidity of a connected series of silos of this type, concrete columns are provided at the corners of the different compartments, as indicated in Fig. 224, and the division walls are securely bonded with the columns by means of strips of expanded metal, about 4 ft. long by 5 in. wide, laid flat at the intersections of the walls and crossing the columns in each direction.

The illustrations here given represent the type of design adopted for an extensive series of silos erected in South

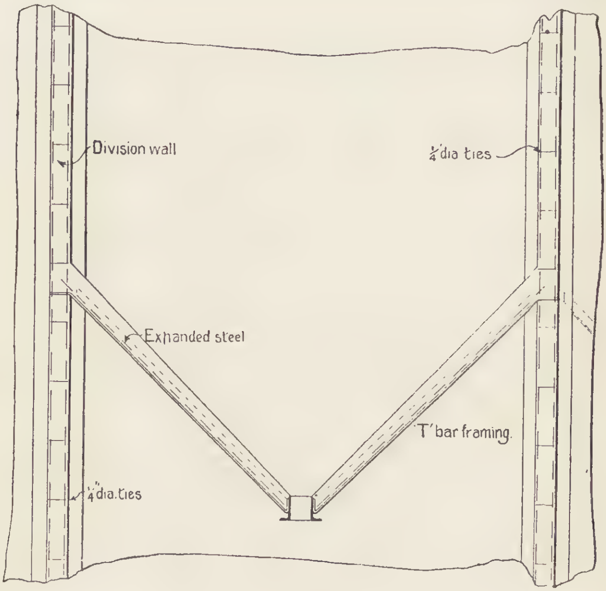


Fig. 225.

America, but they are equally applicable to any requirements, providing the proportions be correctly adapted to the loads determined for any given case.

CHAPTER XI

A Modern Hotel Building

189. General Construction.—The Marlborough-Blenheim Hotel, recently completed in Atlantic City, U.S.A., is a building about 560 ft. long by 125 ft. wide, the front block being twelve storeys high surmounted by a three-storey dome rising to the height of 164 ft. above ground level. The back block is nine storeys high, and rises to the height of about 96 ft. above the ground. Between the front and back portions is a centre block of six storeys 147 ft. long by 45 ft. wide.

The front block has eight sides, exclusive of the towers and projecting wings, and, as may be judged from Fig. 226, is a very handsome and imposing structure. It is surmounted by a central dome flanked by two side towers, and two boiler chimneys treated so as to harmonise with the architectural features of the building.

Before the front block is a crescent-shaped "solarium," of two storeys partly enclosing a spacious piazza.

Along one side of the centre block is the one-storey "sun parlour" shown in the block plan (Fig. 228).

Above the seventh floor the outline of the building commences to change. The octagonal portion of the front block is gradually detached from the centre block, and takes the form of a tower which is surmounted by an octagonal ribbed concrete-steel dome with the diameter of 50 ft. The two side towers are detached from the main building above the eighth floor, where they are surrounded by balconies, and from the flat roof of each rises a domed pavilion.

The entire building was constructed on the Kahn system of reinforced concrete and hollow tile, at a total cost of the

building exclusive of land, equipment, and furniture, of £200,000.

Fig. 227 is a view illustrating the manner in which the

Fig. 226.—Marlborough-Blenheim Hotel, Atlantic City.

cage system of construction has been applied in this building, and Fig. 228 is a block plan showing the general arrangement.

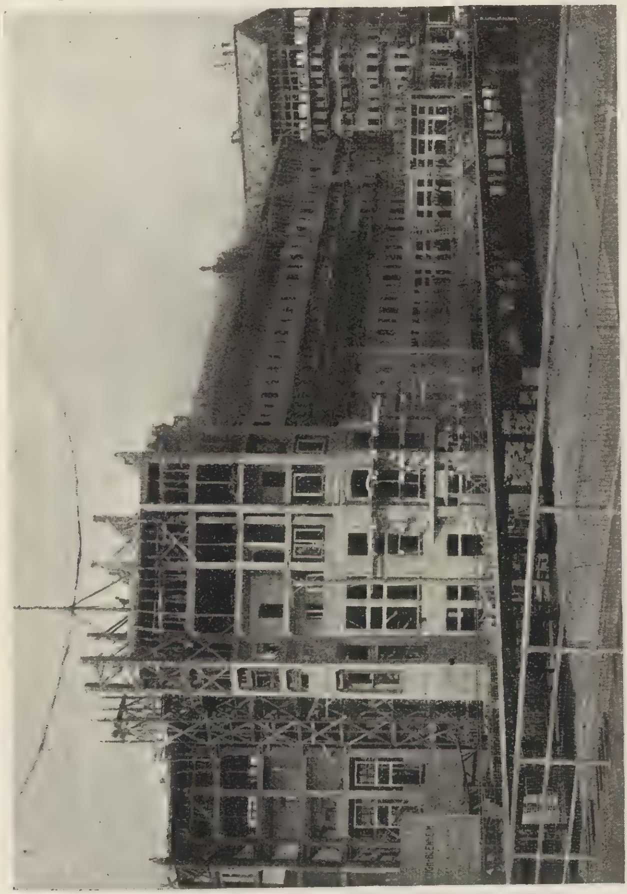


FIG. 227.—View of Hotel during Construction.

190. Foundations.—No general excavation was necessary for foundation work, as the basement is above normal

ground level. Therefore the ground simply required to be levelled for the reception of the basement floor between the column and wall footings.

All the footings are supported on piles averaging 20 ft. long by 8 in. diameter, driven into sand at the rate of twenty an hour by means of a water-jet assisted by occasional blows from hand hammers. Altogether some 1,800 piles were used.

The foundations include 159 rectangular column footings, in addition to footings for the outer walls.

All the column footings are of concrete-steel, generally about 7 ft. square, and carried on nine or ten piles. Those footings which support columns beneath the main dome

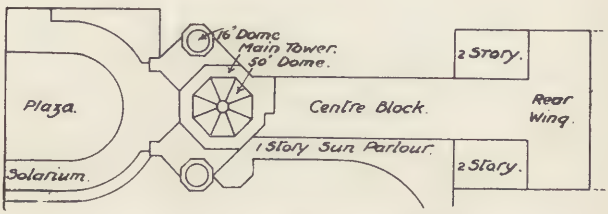


FIG. 228.—Block Plan, Marlborough-Blenheim Hotel.

rest upon twelve piles each, while some of the footings under comparatively light loads only required five piles each.

Rectangular holes were dug to a depth of 12 in. below the level at which the heads of the piles were cut off, and concrete was deposited in each pit to the depth of 27 in. The concrete was mixed in the proportions of 1 part Portland cement, $2\frac{1}{2}$ parts sand, and 5 parts stone, crushed to pass through a $\frac{3}{4}$ in. diameter ring.

In each footing so formed horizontal Kahn bars were embedded at a distance of 2 in. above the top of the piles, and four vertical anchor bars 3 ft. 6 in. long to connect the pier and columns.

Each column footing is made with an upper course of concrete 12 in. high with a square hole in the centre, so

that after the column had been moulded it was connected with the footing by a tenon and mortise joint, as shown in Fig. 229.

The wall and column footings are connected by concrete-steel beams, about 4 ft. below the levelled surface of the ground, as represented in Fig. 230.

In this example the beam measures 22 in. wide by 12 in. deep, and is reinforced by three 2-in. by $\frac{3}{4}$-in. Kahn bars, 18 ft. long, near the lower surface of the concrete, and by two inverted bars of the same dimensions, 9 ft. long, near the upper surface of the concrete.

191. Columns.—The walls and floors are supported by concrete-steel columns, the general dimensions of which are 24 in. square at the base progressively diminished to 12 in. square at the top. The columns are disposed in rows parallel with the sides of the building, and spaced about 18 ft. apart centre to centre.

The square columns are reinforced by vertical bars, generally four in number, connected by diagonal ties from corner to corner. In cases where the loads are unusually heavy, the reinforcement comprises eight vertical bars and three diagonal ties.

Eight columns supporting the octagonal dome are of circular cross section, 24 in. diameter, and each of them is reinforced by four vertical 1-in. diameter bars, with a spiral winding of No. 8 gauge steel wire, wired to the vertical bars, the pitch of the spiral coil being 3 in.

All columns throughout the building are formed with solid knee-braces at their junction with beams and floor slabs, these bracketed extensions being reinforced by bent bars carried from the columns to the floor beam or slab, as the case may be.

In each outer wall of the back block two wall columns, spaced 10 ft. apart centre to centre, support at first floor level a beam 24 in. wide by 42 in. deep with the clear span of 8 ft. This beam carries a 24 in. square wall column extending through six storeys, and supporting a total load of 260,000 lb. Details of the arrangement are shown in Fig. 231.

At the level where the roof of the centre block intersects

the walls of the front block the complicated character of the framework made it necessary to employ various columns in special positions.

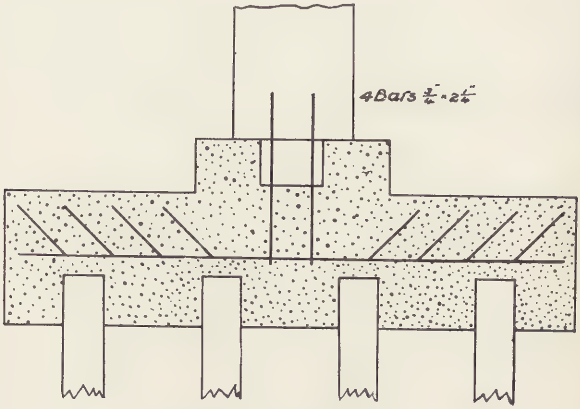


FIG. 229.—Column Foundation.

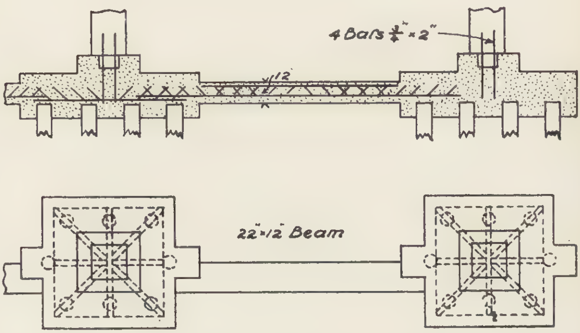


FIG. 230.—Typical Column Foundations.

One of these, above the roof of the centre block, is supported by a concrete-steel girder 18 in. wide by 40 in. deep

connecting two of the ordinary columns. Details of the construction will be found in Fig. 232.

Particulars relative to column moulds and moulding will be found in Articles 202 and 203.

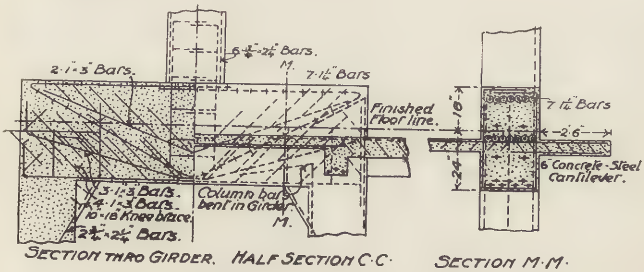


FIG. 231.—Beam supporting Columns in Back Block.

192. General Floor Construction.—As the architect required that no projections should occur below normal ceiling level, other than the wind struts at every alternate row of columns, transverse floor beams could not be employed in the centre and back blocks.

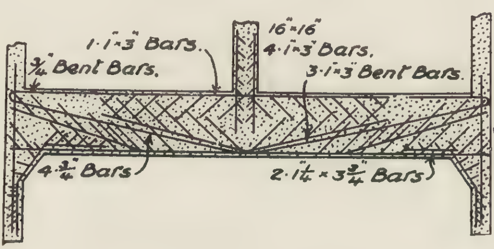


FIG. 232.—Beam supporting Columns in Centre Block.

Consequently the floor panels are of considerable span. The wind struts are beams 8 in. wide by 18 in. deep, flush with the finished floor level, projecting about 6 in. below the ceiling line. The struts are situated between every alternate

row of columns, the average spacing being about 36 ft. centre to centre.

The wall and interior columns are connected by longitudinal beams of concrete-steel, 10 in. wide by 16 in. deep, reinforced by two 3-in. by 1-in. and one $2\frac{1}{4}$-in. by $\frac{3}{4}$-in. Kahn bars and a $\frac{3}{4}$-in. diameter bar 6 ft. long, passing through each column near the surface level of the beams for the purpose of resisting tension due to continuous-girder action.

With the exception of special floor areas, the average span of the floor panels is about 18 ft. centre to centre, the maximum span being 25 ft. The floor slabs are formed of hollow tiles 6 in. square by 12 in. long, laid in rows 4 in. apart perpendicular to the floor beams. The spaces between the rows of tiles are filled in with concrete, which is also spread to the thickness of 2 in. above the tiles.

FIG. 233.—Section of Floor Construction.

By the application of Kahn bars near the lower surface of the filling material a connected series of T-section beams is formed between and over the rows of tiles.

The construction is shown in detail by Fig. 233.

The reinforcement employed throughout the floor beams and slabs consists of Kahn bars, the characteristic of which is represented by two projecting side webs that can be cut and bent up to form stirrups for withstanding shearing stresses or tension on diagonal planes.

As the sizes of the bars vary from place to place according to the load, it would serve no useful purpose to give dimensions of those in isolated beams and slabs.

On the two lower floors of the building the floors are covered with a 2-in. layer of cinder concrete with embedded battens, upon which wood floors were nailed. In the upper storeys the floor slabs are covered with a 2-in. layer of cinder concrete, in which gas, water, and electric light conduit pipes

were embedded, and the floor surface was completed by a 1-in. finish of concrete.

193. Floors under Main Dome. — Beneath the octagonal dome the floor in each of the two lowest storeys consists of a concrete-steel slab supported on the beams connecting the eight main columns.

Having no intermediate supports, these slabs were made 8 in. thick, and reinforced by two series of $1\frac{1}{2}$-in. by $\frac{1}{2}$-in. Kahn bars spaced 6 in. apart at the centre and 12 in. apart near the edges. The bars were crossed to form a network capable of withstanding tensile stresses in every direction.

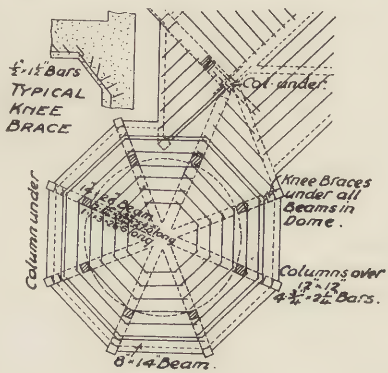


FIG. 234.—Plan of Beams in Side Towers.

Moreover, the octagonal slabs were bonded with the adjoining floor panels by monolithic connection of the concrete, and by $1\frac{1}{2}$-in. by $\frac{1}{2}$-in. bars 8 ft. long spaced 2 ft. 8 in. apart, placed in an inverted position near the top surface of the concrete to withstand tensile stresses arising from continuous-girder action in the slabs.

These slabs were built with the camber of 1 in. to allow for possible sagging on removal of the centering.

On all other storeys beneath the dome the octagonal floors are of tile and concrete-steel construction, supported by four girders extending across and intersecting at the centre of the octagon.

The tiles used were 3 in. square and 4 in. square, according to the load, being laid in rows a short distance apart, and reinforced by Kahn bars placed in the concrete at intervals of 1 ft. 4 in. The slab was completed, as described in Article 192, by a 2-in. layer of concrete.

The radial beams measure 12 in. wide by 20 in. deep, and each of them is reinforced by two $3\frac{3}{4}$-in. by $1\frac{1}{4}$-in. bars 32 ft. 6 in. long, and one $2\frac{3}{8}$-in. by $\frac{3}{4}$-in. bar 15 ft. long.

194. Floors under Side Domes.—Beneath the side domes all floors up to the eighth storey are continuous with the main floors of the central tower, and are supported by four intersecting girders carried on eight columns, the tops of the columns being connected by circumferential beams (see Fig. 234).

These girders are not supported in any way at the point of intersection, and carry the upper columns of the tower, shown by hatching in Fig. 234 at a distance of 4 ft. inside the outer ring of columns below.

The floor slabs in these towers are reinforced with $1\frac{1}{2}$-in. by $\frac{1}{2}$-in. bars, 16 in. apart, parallel with the walls of the octagon.

195. Dome Construction.—In the central dome the tops of the eight columns are connected by horizontal girders forming a frame from which spring the ribs of the dome. A plan and section of the main dome are given in Fig. 235.

The dome shell consists of 5-in. cinder concrete slabs, 5 in. thick, reinforced by radial and circumferential bars 1 in. from the inside surface. The main ribs project beyond the shell both inside and outside, and are carried up to the lantern above. This lantern is formed with a vertical wall of cylindrical form, 6 ft. high, and is surmounted by a hemispherical solid concrete roof of 14-ft. diameter.

The side domes are carried on columns supported as described in Article 194, and connected at the top by a ring beam $13\frac{1}{2}$ in. wide by 14 in. deep, and having an internal diameter of 14 ft. 9 in. Each dome has the internal diameter of 16 ft., and is built of concrete reinforced as illustrated in Fig. 234 by radial and horizontal bars. The concrete was mixed in the proportions of 1 part Portland cement, $2\frac{1}{2}$ parts sand, and 5 parts anthracite cinder.

196. Walls.—All outer walls are built of hollow tiles 12 in. thick up to the first floor and 8 in. thick for all floors above. The tiles are laid in Portland cement, the exterior surface being rough cast with small shingle in 1 : 1 Portland

cement mortar. The inner face of the walls is plastered and decorated to suit the character of the rooms.

The outer wall surface is built up to the exterior face of the columns, which project inside the building as pilasters except where partitions or other interior fittings occur.

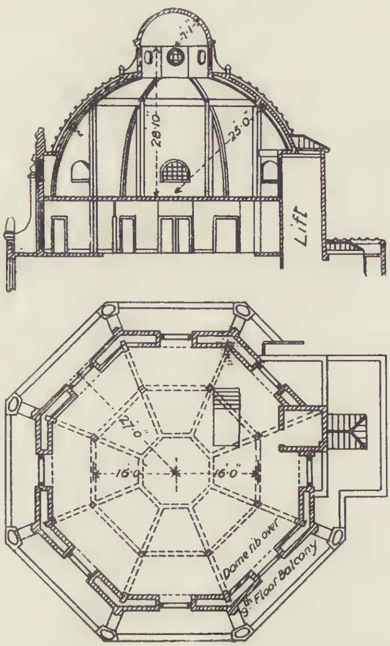


Fig. 235.—Section and Plan of Main Dome.

The wall columns are connected at the different storeys by horizontal beams common to the wall and floor systems. All the interior partition walls were arranged in accordance with architectural requirements, and built of hollow tile as in the case of the outer walls.

197. Roof Construction.—In this Article particulars

are given relative to typical and special details of roof construction. The main roofs of the front and back buildings are carried by the upper wall beams, which are analogous to wall-plates, and by two lines of beams connecting the tops of the interior columns at a higher level, these beams being equivalent to purlins. The ridge in each case is parallel with the axis of the building. Between the upper and lower horizontal beams inclined rafters of concrete-steel are spaced about 12 ft. apart, but there are no rafters between the upper horizontal beams and the ridge. Fig. 236 is a

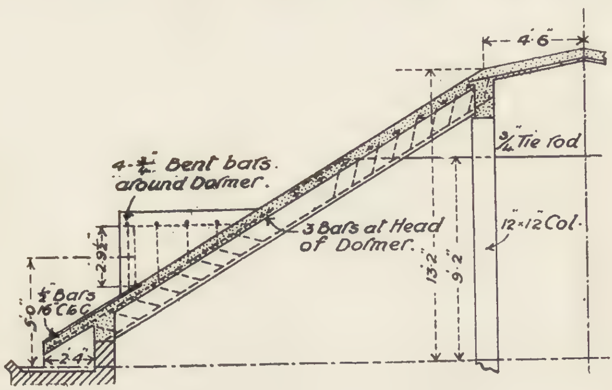


FIG. 236.—Half-Section of Roof (Centre Block).

typical half-section of the construction, where it will be seen that the arrangement of the framework is somewhat similar to that of a queen-post roof.

All the roof members are connected by a concrete-steel slab flush with the upper surface of the rafters, and formed of cinder concrete mixed in the proportions of 1 part Portland cement, $2\frac{1}{2}$ parts sand, and 5 parts anthracite cinder reinforced by $1\frac{1}{2}$-in. by $\frac{1}{2}$-in. Kahn bars 16 in. apart centre to centre. To take the thrust of the upper portion of the roof, a horizontal rod of $\frac{3}{4}$-in. diameter connects the beams beneath the head of each pair of rafters.

Large and small dormers are built between the rafters.

There is one large dormer at each side of the centre block (see Fig. 1). All the dormers have continuous concrete-steel slabs, forming roof and walls without beams or struts.

In the small dormers (see Fig. 236) the reinforcement consists of $\frac{3}{8}$-in. diameter bars 12 in. apart, each bar being curved at the top to suit the configuration of the roof, and terminating in two vertical legs for reinforcement of the side walls.

In the large dormers the reinforcement consists of $1\frac{1}{2}$-in.

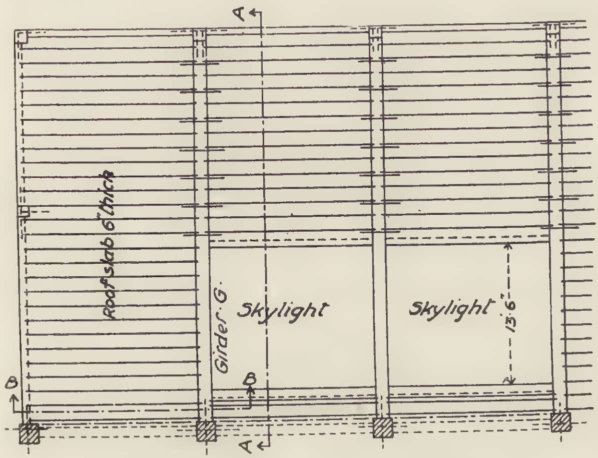


FIG. 237.—Plan of Roof over Banqueting Hall.

diameter bars spaced 12 ft. apart, parallel with the axis of the building.

The main roofs and dormers are covered with tiles secured by copper nails.

Fig. 237 is a plan showing part of the flat roof over the banqueting hall on the ground floor of the back building. The three girders, each 16 in. wide by 36 in. deep, have the clear span of 36 ft. They project 16 in. above and 14 in. below the roof slab, which is continuous with the slab of the first floor. The roof slab is formed of 4 in. hollow tiles laid

in rows with a 2-in. layer of concrete on the top, the concrete between the rows being reinforced as described in Article 192.

Figs. 238 and 239 contain details of the construction.

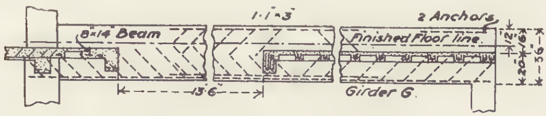


FIG. 238.—Section AA (see Fig. 237).

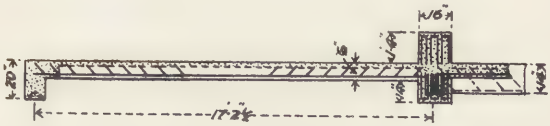


Fig. 239.—Section BB (see Fig. 237).

198. Roof Cornice. —Fig. 240 illustrates a portion of the main roof cornice, formed by a cantilever extension of the upper floor slab for a distance of 3 ft. 10 in. beyond the

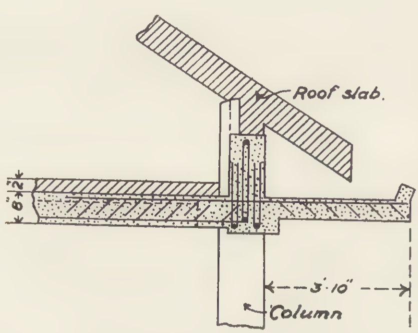


FIG. 240.—Section of Main Roof Cornice.

outer face of the wall, and made with a curb so as to form a rain-water gutter.

The slab is built of 4-in. tiles with a 2-in. layer of concrete above, and is reinforced by Kahn bars in an inverted position near the upper surface of the concrete. The sloping roof slab is here supported by a longitudinal beam 10 in. wide by 28 in. deep connecting the wall columns. Anchor bars, of $\frac{3}{8}$-in. diameter and projecting 12 in., were built into the wall beams at intervals of 2 ft. 8 in. apart for the connection of the roof slab.

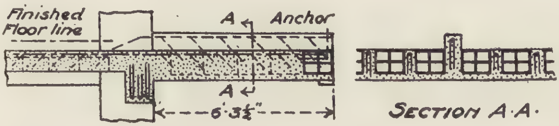


FIG. 241.—Typical Sections of Balcony Construction.

199. Balconies.—As may be seen by reference to Fig. 226, numerous balconies project from various parts of the buildings. The floor of all the balconies are formed by cantilever extensions of the main floor slabs, projecting from 2 ft. 6 in. to 6 ft. beyond the outer wall surface.

In the narrower balconies the slab is not stiffened by beams, but those of greater span receive support from cantilever beams 6 in. wide by 18 in. deep at intervals corresponding with the spacing of the wall columns, and projecting above the floor surface as shown in Fig. 241.

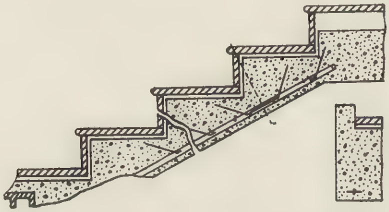


FIG. 242.—Part Section of Main Stairs.

200. Stairways.—Fig. 242 is a section illustrating the construction of the main stairway.

All stairs throughout the buildings are formed with an inclined lower surface, the concrete being reinforced by $1\frac{1}{2}$-in. by $\frac{1}{2}$-in. bars 16 in. apart, and at a distance of 1 in. above the lower surface. The risers are finished with a layer of cement $\frac{1}{2}$ in. thick, covered by slabs of marble.

In the octagonal portion of the front block the main stairway occupies six sides of the octagon, leaving a spacious central well.

201. Fire Escape Tower.—At one side of the centre block there is a tower, measuring 11 ft. 9 in. by 7 ft. 4 in., extending to the roof to afford outside communication between every floor and the ground. With the object of securing the complete isolation of the tower no openings have been made into it from the main building, access being afforded at every storey by a door which can be reached by a cantilever balcony.

202. Concrete Moulds. — Timber moulds for all concrete-steel members were made on the site, the saw-mill being driven by an electric motor, which was also used to operate one of the concrete hoists.

Fig. 243 illustrates the construction of the column moulds, and Fig. 244 that of the beam and floor slab moulds.

The column moulds were formed of four 2-in. boards, clamped as shown in the drawing. Hand holes were provided at the bottom to facilitate adjustment of the bars before the deposition of concrete.

The girder moulds were simple troughs supported on posts, and between them were built platforms for the tile and concrete floor slabs, supported by posts suitably braced.

203. Concreting.—Before the commencement of concreting the columns, the wings projecting from the sides of the Kahn bars used as vertical reinforcement were bent to a horizontal position at the end of each bar, and interlocked with those of the bars in the column lengths below, and at the top they were fixed by strips of wood nailed to the moulds. Concrete was then tipped in at the top of the column moulds and well poked between the reinforcing bars.

Concrete was next deposited in a layer $1\frac{1}{2}$ in. thick over the bottom of the beam moulds. On this the reinforcement was laid and the moulds were filled up.

The rows of tiles for the floor slabs were laid out on the

platforms at the predetermined distance apart, and 1-in. layers of concrete were deposited between the rows. On this the reinforcing bars were placed, the spaces were filled to the top level of the tiles, and the whole surface was covered with a 2-in. layer of concrete extending also over the tops of the columns and beams.

As the supports beneath the beams and floor slabs were left in position for three weeks after the completion of concreting, four complete sets of moulds and supports were required for the uninterrupted conduct of work.

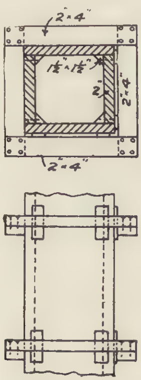


Fig. 243.
Column Moulds.

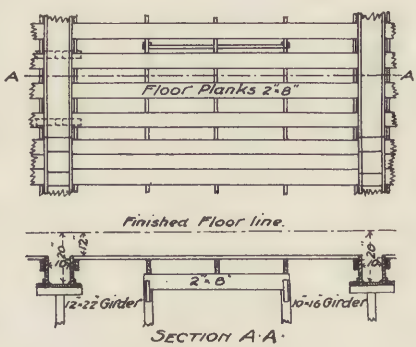


Fig. 244.
Beams and Floor Moulds.

The outer walls and interior partitions were built up simultaneously with the columns and floors. Thus the building grew up storey after storey without the necessity for scaffolding.

That part of the main roof above the rafters (see Article 197) was moulded on a platform, the concrete being spread and trowelled level.

All concrete was delivered to the upper storeys by two hoists, and tracks were laid as necessary for the wheelbarrows in which the material was conveyed to different points.

Each floor covers an area of about 30,000 sq. ft., and

was concreted in 3 days, work being commenced simultaneously from each end. The time occupied in erecting the moulds, concreting the columns, beams, and floors, building the walls, laying the floor tiles, and removing the moulds for re-erection was two weeks per storey.

204. Storehouse and Concrete Plant. — Cement used during the construction of the buildings was stored in a weather-proof storehouse with a capacity of 3,000 bags. Broken trap rock, cinders, and sand were stored in heaps on the site.

All concrete was prepared in a Campbell and a Foote mixer, each driven by belting from a steam engine. The concrete was mixed in batches of 13½ cubic feet, and distributed by wheelbarrows on plank tracks to the piers at all parts of the site, and was raised to the upper floors by the hoists mentioned in Article 203, and distributed by wheelbarrows as at ground level.

205. Concrete Data.—Most of the concrete was mixed very wet, to permit its thorough penetration to all parts of the moulds and between the bars of the reinforcement. The proportions and consistency of the concrete used are given in the subjoined table :—

Reference.	Class of Work.	Proportions of Concrete.			Consistency.
		Portland Cement.	Sand.	Aggregate.	
		Parts.	Parts.	Parts.	
Art. 190	Foundations.	1	2½	5, trap rock crushed to pass through a ¾-in. diameter ring.	Medium.
,, 191	Columns .	1	2½	5 ,, ,,	Very wet.
,, 192 to ,, 194	Beams .	1	2½	5 ,, ,,	,,
	Floor and other slabs .	1	2½	5 ,, ,,	,,
,, 195	Dome shells .	1	2½	5, hard anthracite cinder.	,,
,, 197	Roof slopes .	1	2½	5 ,, ,,	Stiff.

206. Hollow Tiles.—All the hollow tiles used in the floors, walls, and other details of the building were made by the National Fireproofing Company, of New York. They measure 12 in. square, and vary in depth from 3 in. to 12 in.

The following schedule gives some data relative to their physical properties :—

Average weight	40 lb. per cub. ft.
Crushing strength, in small cubes	4,600 lb. per sq. in.
Crushing strength, full size—	
Sideways	2,020 lb. per. sq. in.
Endways	4,000 ,, ,,

207. Reinforcement.—With the exception of some round bars, rods, and wire, all the reinforcing bars were of the Kahn pattern, as supplied in Great Britain by the Trussed Concrete-Steel Company, of Westminster. Fig. 245 includes a cross section and a perspective illustrating the form of these bars, which are made to the following standard dimensions :—

Size of Bar.	Length of Diagonals.	Sectional Area.	Weight per Foot-lb.
		Sq. In.	Lb.
$1\frac{1}{2}$ in. by $\frac{1}{2}$ in.	6 in. and 8 in.	0.38	1.4
$2\frac{3}{16}$,, $\frac{3}{4}$,,	8 in. and 12 in.	0.78	2.7
3 ,, 1 ,,	12 in., 18 in. and 24 in.	1.42	4.8
$3\frac{3}{4}$,, $1\frac{1}{4}$,,	18 in., 24 in. and 30 in.	2.00	6.9

FIG. 245.—The Kahn Trussed Bar.

208. Column and Floor Loads. — The maximum regular column loads were 296,000 lb. each. The main

dome columns were proportioned for a load of 353,000 lb. each.

The floors of the entrance lobby and of a large hall on the first floor in the front block were designed for a live load of 75 lb. per square foot, all the other floors for a live load of 50 lb. per square foot, and the roofs for a live load of 30 lb. per square foot.

209. Unit Stresses.—The compressive strength of the concrete was taken at 2,400 lb. per square inch. The unit stresses allowed were—

500 lb. per sq. in. for dead and live loads.
750 „ for wind loads.

For the steel reinforcement the unit stresses allowed were—

16,000 lb. per sq. in. for dead and live loads.
20,000 „ for wind loads.

In designing the floors no account was taken of the resistance of the hollow tiles, the concrete-steel being made adequate to withstand all compressive and tensile stresses developed.

210. Quantities of Materials Used.—Particulars relative to the quantities of the chief materials used in the construction of the building will be found in the subjoined schedule—

Piles, 8-in. diameter	1,800
Timber for moulds and supports	500,000 ft.
Stone concrete (apart from cinder concrete)	10,000 cub. yds.
Hollow tiles	132,680 cub. ft.
Steel as reinforcement	550 tons.

211. Architects and Contractors.—Messrs. Prince & M'Lanahan, of Philadelphia, were the architects, and the general building contractors were Messrs. E. Gilbert & Co., of the same city.

All structural designs for the work described in this chapter were prepared by the Trussed Concrete-Steel Com-

pany, of Detroit, for the National Fireproofing Company, of New York, who were sub-contractors for the concrete-steel and tile construction.

About 300 men were employed in the execution of the works.

CHAPTER XII

THE RENOMMÉE HALL, LIÉGE—CHATEAU D'EAU, PARIS —A BANK BUILDING, PARIS—CONCERT HALL, STRASBURG—POPULAR THEATRE, MUNICH

THE RENOMMÉE HALL, LIÉGE

212. Main Features.—This fine building was built entirely of concrete-steel by the firm MM. Perraud & Dumas, of Brussels, from the designs of M. Paul Jaspar, of Liége.

The general arrangement, the style and proportions of all the parts of the building, were specially settled by the architect, so as to be the most suitable for concrete-steel construction, and in order to use the properties of that material to the best possible advantage. One great object was to avoid employing concrete merely as an imitation stone, by adopting a characteristic design indicating the nature of the material actually used. The success attained in this direction may be realised by inspection of the interior view shown in Fig. 246.[1]

213. Principal Hall.—As may be seen by Fig. 247, the principal hall is covered by three cupolas, each 55 ft. diameter, placed at a height of about 50 ft. above ground level. Each cupola forms part of a sphere which is continued in haunches pierced with lights and descending to the corners of a circumscribed square. The intersections of the spheres with the vertical planes passing through the sides of the squares are formed by arched ribs which spring from the capitals of short cylindrical columns. The

[1] The particulars and illustrations of this building have been taken by permission from the Proceedings of the Institution of Mechanical Engineers for 1905.

cupolas are $4\frac{1}{2}$ in. thick, and are made of clinker concrete reinforced with expanded metal and a latticed arrangement of steel bars.

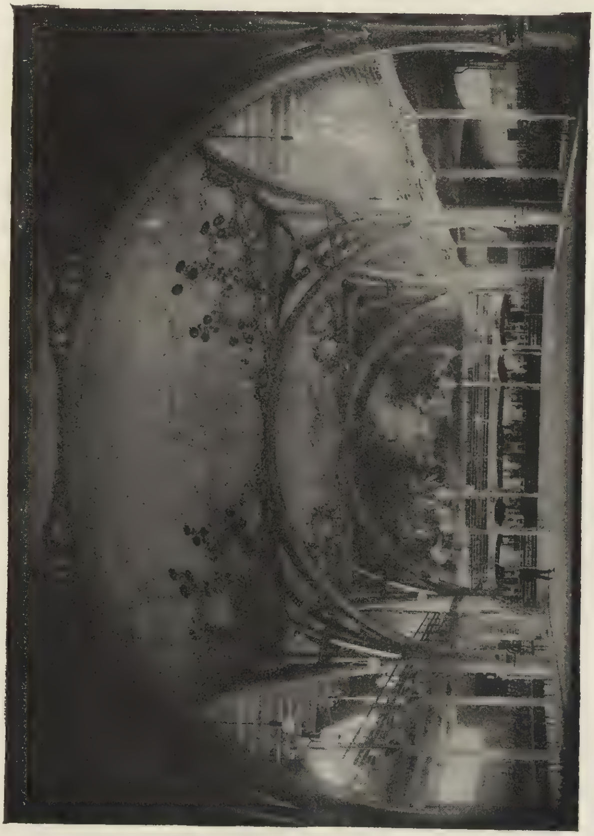


FIG. 246.—Interior View, Renommée Hall, Liége.

214. Lighting of Principal Hall.—The principal hall is lighted at the sides by six semicircular glass lights, each 52.5 ft. diameter, framed by arched beams. The spandrils

are formed by concrete-steel panels, on the inside of which are fixed ornamental designs in relief. The moulding of

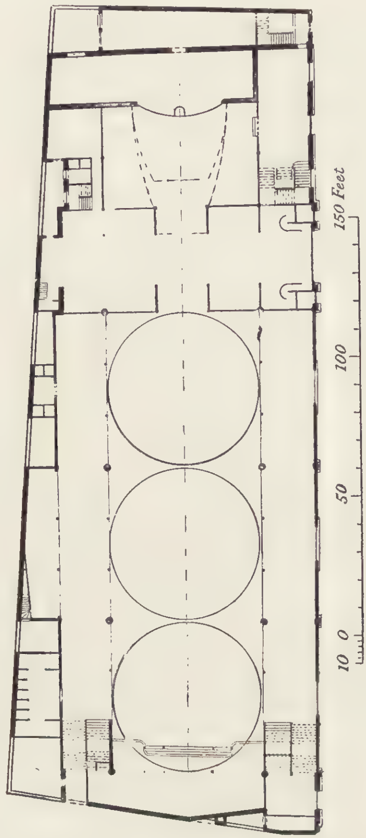


FIG. 247.—Plan of Renommée Hall, Liége.

these decorations was performed in the workshop, where each panel was cut into portions that were afterwards erected in position on the site.

215. Novel System of Centring.—The centring of

the first cupola built embodied some distinctly novel features. In order to avoid the great expense entailed by the construction in timber of a spherical centring, a skeleton was built up of ironwork consisting of 16 bars, each of $1\frac{1}{2}$-inch diameter, fixed upon meridian lines like the ribs of an umbrella, and interlaced upon parallel horizontal circles by other bars of smaller diameter. The whole skeleton was then covered with sheets of expanded metal, designed to act as the first reinforcement, on which the concrete was afterwards placed above and below so as to completely surround the expanded metal, which thus acted as its own centring, and it was merely necessary to render the surface up to the required thickness. It was intended that the bars of the skeleton should be similarly used for the other cupolas, and finally for reinforcing the beams.

Unfortunately, this system of centring was found to be wanting in rigidity, and it was necessary after all to make use of timber.

216. Terrace Roof.—The roof of the galleries and the spherical triangles between the cupolas form a terrace of 957 square yards area, which serves as a promenade. The concrete of the cupolas and the terraces is covered by a layer of ruberoid as an additional precaution against the penetration of moisture.

In spite of the complete absence of ornamental mouldings, which were left out to facilitate the centring, the hall is of most elegant appearance, and reflects much credit upon the architect and contractors alike.

CHATEAU D'EAU, PARIS

217. Main Features.—The Chateau d'Eau at the Paris Exposition of 1900 may be mentioned as a remarkable example of construction on the Coignet system. That part of the structure which excited more attention than any other was the great alcove forming the principal façade. The alcove, of which Fig. 248 is a view, measured 45 metres high by 25 metres wide over all, and was supported upon a series of concrete-steel curved walls only 10 centimetres thick, carried by the reinforced construction of the

galleries and staircases in the lower part of the building. The semicircular wall forming the lower portion of the interior of the alcove was also only 10 centimetres thick, but provided with vertical stiffening ribs 20 centimetres square. The upper portion of the alcove was composed of arched ribs covered by a concrete-steel slab 6 centimetres thick. The outer face consisted of two

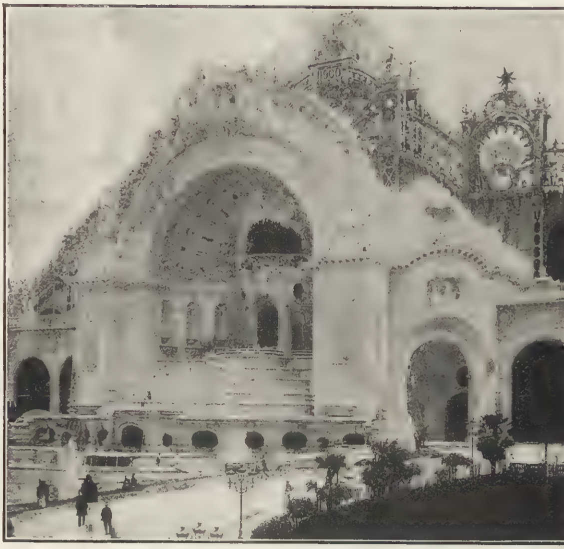


FIG. 248.—Chateau d'Eau, Paris.

concentric semicircular arched ribs connected by bracing and faced with reinforced concrete.

A BANK BUILDING, PARIS

218. General Construction.—Fig. 249 represents the exterior of the Banque des Valeurs Industrielles, Paris, built by M. Edmond Coignet. A transverse section of the building is given in Fig. 250, where the reader will see that it

comprises nine storeys in all, including the basement and sub-basement. Owing to the desire of the architect to

FIG. 249.—Banque des Valeurs Industrielles, Paris.

comply with customary methods of construction, the outer walls are of brick and stone, and the two lowest floors of brick supported on rolled steel joists. The reinforced

concrete work commences at the ground floor, and includes the interior columns, beams, floors, and roof.

219. Floors and Roof.—Fig. 251 is a perspective, and Fig. 252 is a section, illustrating the construction of a

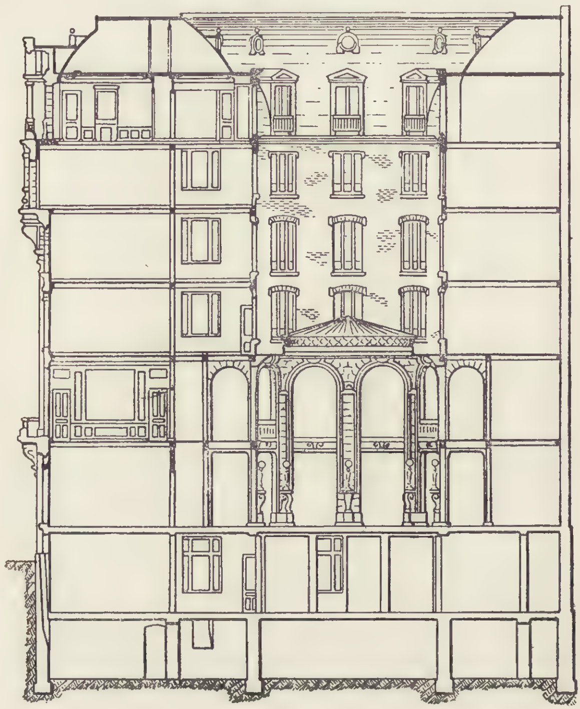


FIG. 250.—Tranverse Section.

Coignet floor beam and slab. The reinforcing bars in the tension area of the beam here represented are 2 centimetres in diameter, and the bars in the compression area have a diameter of 1.3 centimetres. The vertical stirrups, formed of round rods, are lapped round the horizontal bars, and are intended to resist shearing stresses.

In this example the floor slab is reinforced by bars perpendicular to the reinforcement of the beam, and by transverse rods for the purpose of distributing the stresses over these in a uniform manner. The points at which the various bars and rods cross are securely connected by a binding of annealed wire, and the whole of the reinforcement is firmly held together by the surrounding concrete. In this way a complete network is formed capable of resisting stress in every direction. It will be seen that the single bars forming the reinforcement of the floor slab

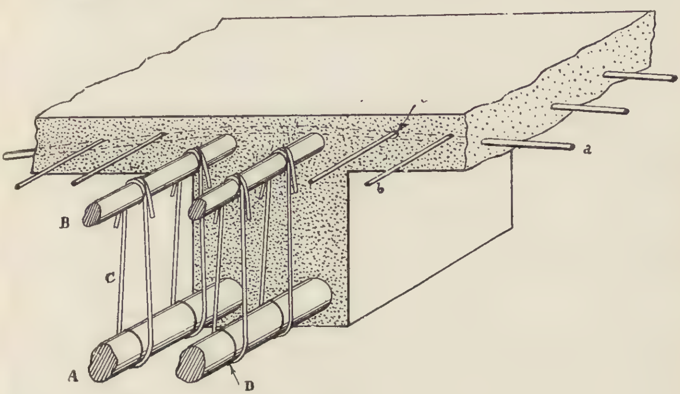


FIG. 251.

are raised so as to pass over the upper reinforcement of the beam, instead of being carried straight through. This arrangement is very desirable in floor slabs where single reinforcement is employed, for the reason that continuous-girder action is always evidenced to some extent, and by bending up the ends of bars terminating in beams, or by bending up bars running across beams, as in the present case, the requisite resistance is offered to tensile stress developed in those parts of the upper area lying between the abutment and the points of contrary flexure. The bars in the floor slab running parallel to the main reinforcement of the beam are also bent up at any

places where they may have to cross floor joists. Floors of this type were employed for spans not exceeding 12 ft.

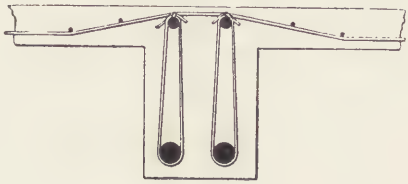


FIG. 252.

and for loads of not more than 56 lb. per sq. ft. The thickness of the floor slabs varies according to requirements from 5 centimetres to 20 centimetres.

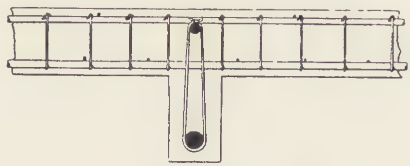


FIG. 253.

In some parts of the building where the spans were considerable the floor slabs were made with double

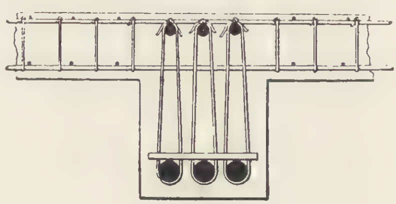


FIG. 254.

reinforcement. Figs. 253 and 254 include sections of floor slabs with double reinforcement.

In some of the longest spans the floor beams were pro-

vided with three sets of double reinforcement, and for the

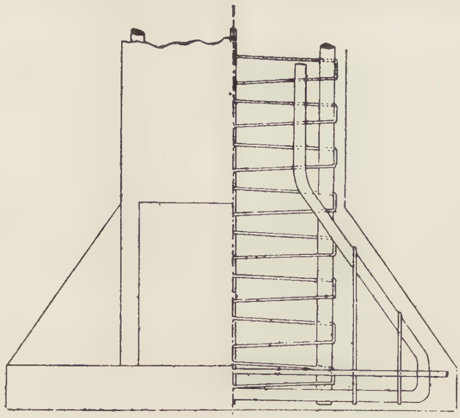


FIG. 255.—Column Details.

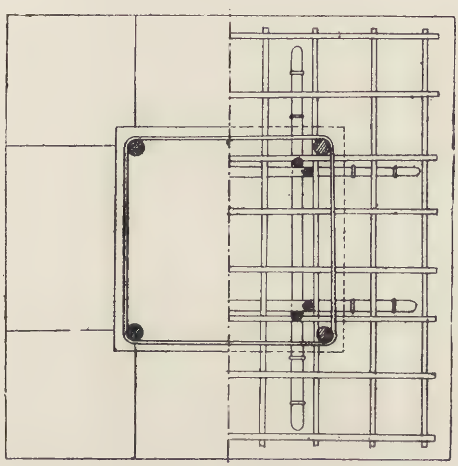


FIG. 256.—Column Footing.

purpose of ensuring even distribution of stress over these bars short transverse rods were fixed (see Fig. 254).

Figs. 255 and 256 are drawings illustrating the reinforcement in the columns and column footings.

All the roofing system of the building is in concrete-steel, designed in a manner practically the same as that followed in the case of the floor slabs. The flat top of the roof is supported in part by columns, and to prevent the admission of moisture the whole of the roof was covered by a layer of waterproof material placed over the concrete.

CONCERT HALL, STRASBURG

220. Main Features.—This fine building contains a grand concert hall, café, restaurant, and club.

It covers a triangular site, with the area of nearly 1,700 square metres. The architects were Messrs. Kuder & Müller, and the building contractor was Mr. E. Züblin, licensee under the Hennebique patents. Some idea of the accommodation provided will be obtained by inspection of Figs. 257 and 258, the former being a longitudinal section and the latter a half plan at first floor level.

The grand concert hall (grosser konzertsaal) is bounded on one side by the outer wall of the building and on the three other sides by the various departments mentioned above. The two principal façades are on Phalsburgstrasse and Jullianstrasse, the angle at the junction of these two streets being occupied by a polygonal tower. The main entrance is in Jullianstrasse, and comprises three large bays giving access to a spacious vestibule. where the booking and pay offices are situated, and thence to a hall providing accommodation for cloakrooms and extending over the whole area beneath the concert hall (see Fig. 257). A garden, where concerts are given, is in communication with the ground floor of the building. At the end of the lower hall are the kitchens (hauptküche and gemüseküche) and scullery (aufwaschküche), service-room (anrichte) of the café and restaurant, the latter establishment facing Phalsburgstrasse. The kitchens also provide for the requirements of a large restaurant and refreshment buffet

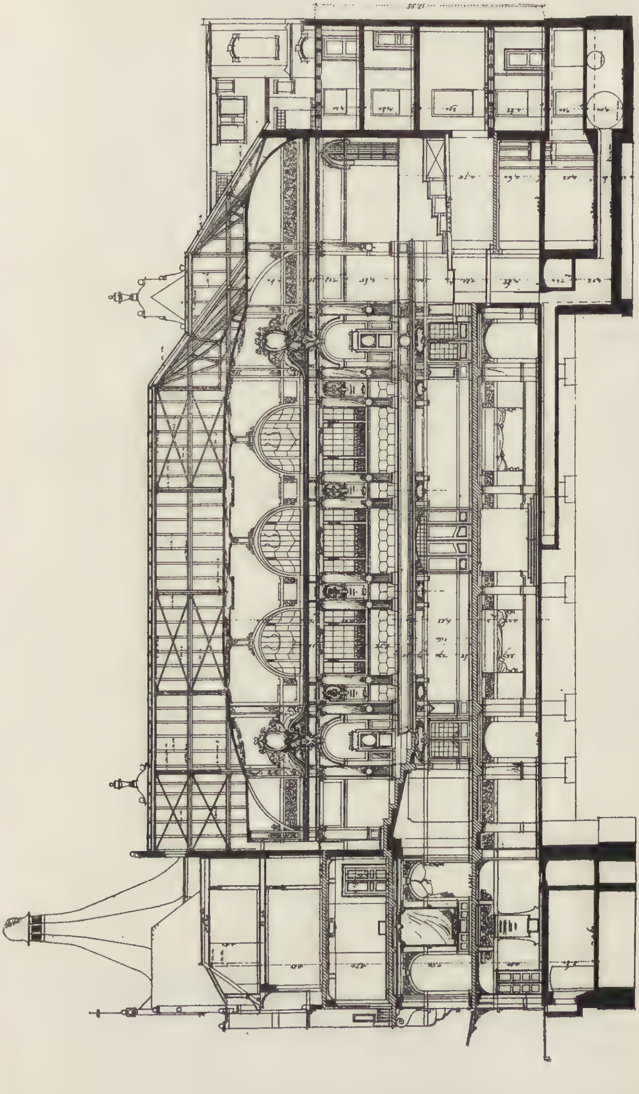


FIG. 257.—Longitudinal Section, Concert Hall, Strasburg.

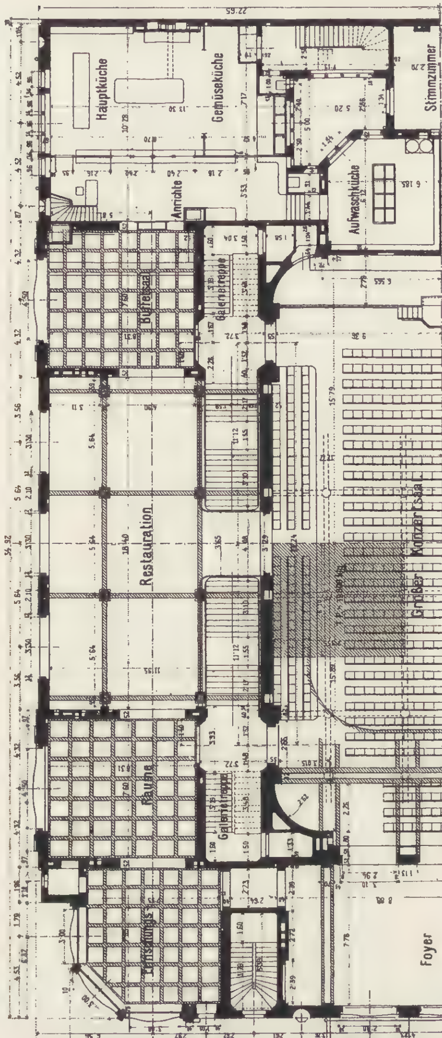


FIG. 258.—Half Plan at First Floor Level.

on the first floor. On the same floor is the grand foyer, 17.50 metres by 7.80 metres, adjoining which is a small foyer to the right.

The grand concert hall comprises a nave 13 metres high, and measuring 32.50 metres long by 19.85 metres wide exclusive of the stage, and two aisles forming galleries along each side of the hall, each of the latter being 18.40 metres long by 3.40 metres wide. The springings of the arched clerestory on which the roof is built are supported by groups of columns. The aisles are divided in their height by a floor which is extended as a balcony projecting 2 metres into the main hall, which contains accommodation for nearly 2,000 persons, and the stage has a width of 12.60 metres and a depth of 9 metres, including the proscenium.

On the second floor, which is at the level of the galleries at either side of the concert hall, are the apartments occupied by the club and the offices of the manager. The two galleries are connected by a balcony at one end of the hall.

Four wide flights of stairs lead from the cloakroom up to the concert hall, and four other staircases rise to the floor on which the galleries are situated, thus providing ample means for the rapid emptying of the entire building.

The greatest difficulty presented in the realisation of the general plan was that the main and partition walls of the upper storeys were necessarily situated so that no support was given by similar members on the storeys below. Therefore it became necessary to design each floor as a huge platform capable of supporting the wall and partition loads of the floors above. The architects decided that concrete-steel construction would lend itself better than any other system to the solution of the problem, and prepared their designs in accordance with the Hennebique system of ferro-concrete.

221. Floor and Gallery Construction.—Fig. 259 is a section showing the construction of the galleries in the concert hall.

The notion that the presence of concrete-steel must necessarily be hidden by false ceilings and other casings is

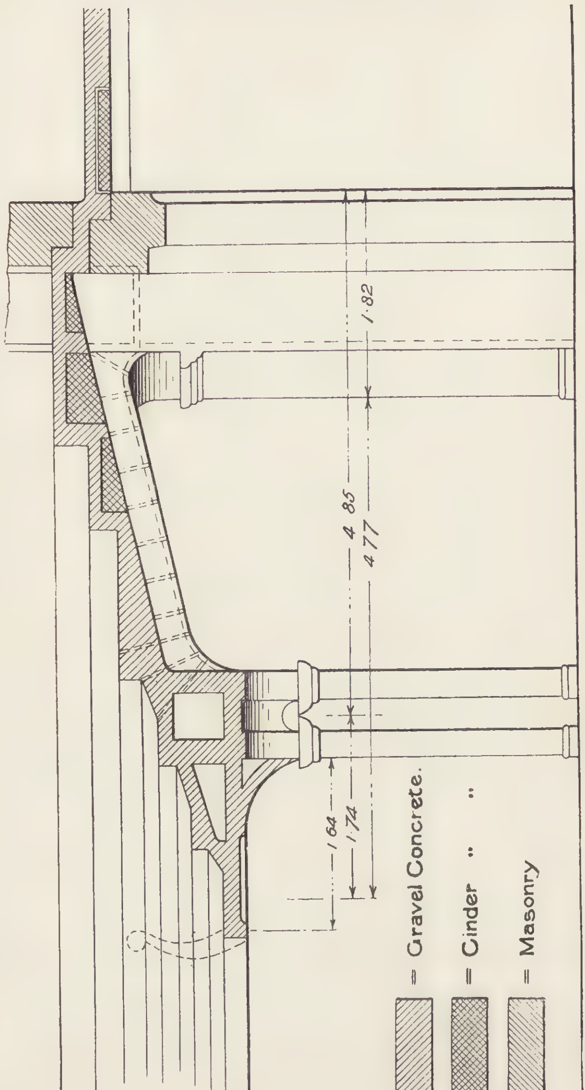


FIG. 259.—Gallery Construction.

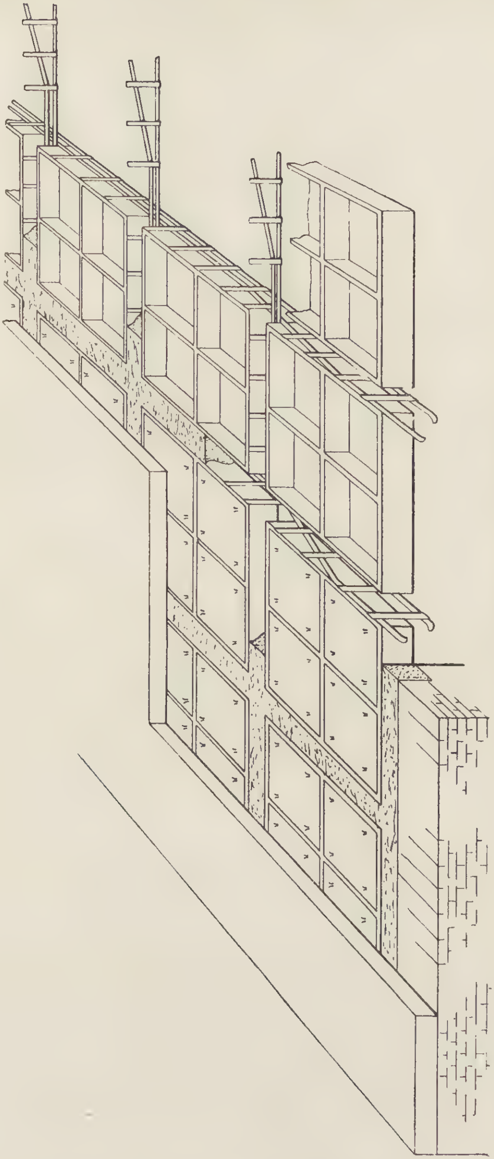


FIG. 260.—Züblin Floor Construction.

being gradually broken down, and architects are beginning to realise that concrete-steel members need no longer be hidden, providing their exterior surfaces receive appropriate decorative treatment.

Thus in the concert hall of Strasburg the beams which constitute the ribs of the floors are plainly revealed, and form ribbed ceilings of entirely satisfactory appearance. Similarly, the galleries projecting boldly from their supports are sustained simply by light and graceful cantilevers, the form of which is not disguised in any way.

222. Züblin Floor Panels.—Fig. 260 shows the construction of the floor, in hollow panels on the Züblin system, which constitutes the ceiling of the foyer, and is prolonged to form one of the balconies on the façade of the building.

223. Floor Loads and Tests.—The floors were calculated for a superload of 400 kilogrammes per square metre. Selected parts of the construction of different spans were tested on completion of the work by loads varying from 400 to 500 kilogrammes per square metre, with the result that the deflection was found to be very small, and nowhere reached the amount of 2 millimetres.

POPULAR THEATRE, MUNICH

224. General Description.—Upon the site of the former building in the Josefspitalstrasse, Munich, a new theatre has been erected from the designs of Mr. Charles Tittrich, by Messrs. Tank Brothers, licensees under the Hennebique patents. The old theatre was condemned on account of the serious risks to which audiences were exposed by inadequate protection against fire, and by insufficiency of the corridors and exits for clearing the house.

With the object of providing adequately for the safety of the public the front portion of the building was separated from the houses on either side. The main entrance on the street façade affords access to the vestibule and box office, while the auditorium and the stage are situated in the space behind. A block plan of the building is given in Fig. 261, and a longitudinal section in Fig. 262.

For reasons connected with the position and construction of the adjoining property, it was not possible to carry the

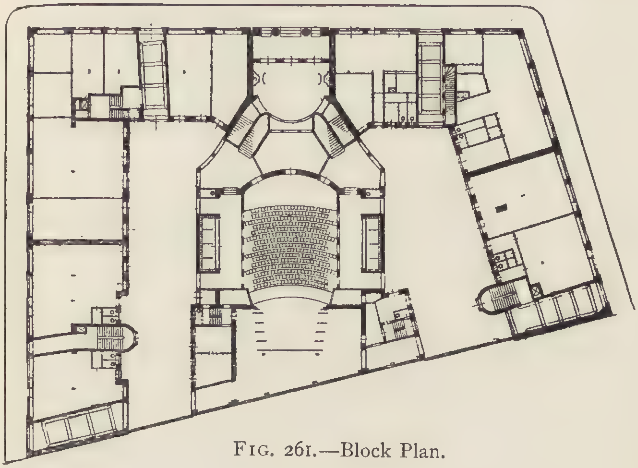


FIG. 261.—Block Plan.

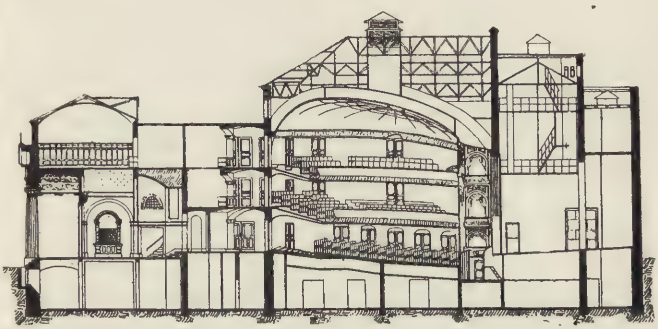
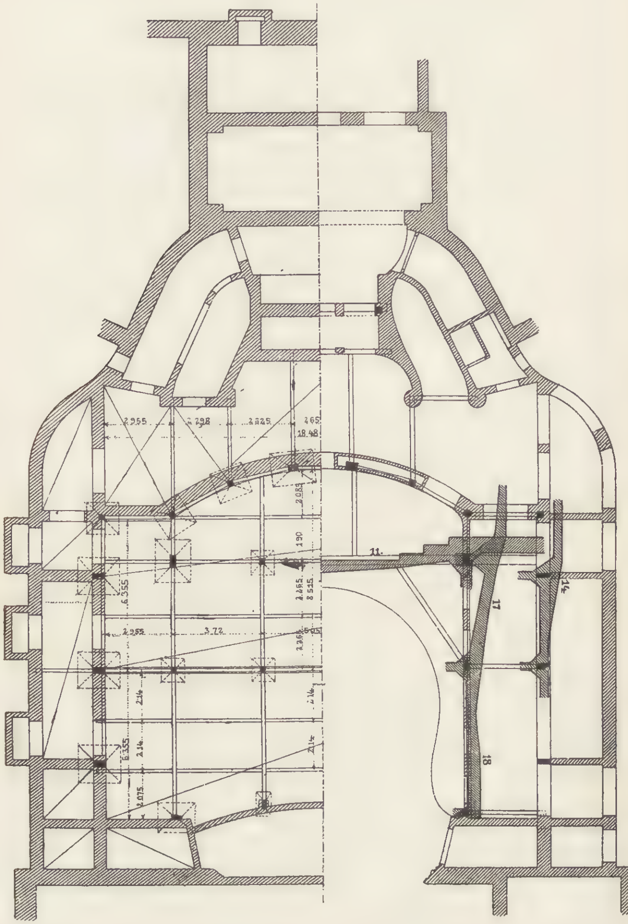


FIG. 262.—Longitudinal Section.

New Popular Theatre, Munich.

theatre to any great height. The auditorium contains merely a parterre and two balconies.



Half Plan of Basement. Half Plan of First Balcony.

FIG. 263.—New Popular Theatre, Munich.

As the parterre is nearly at the same level as the court-yard outside, provision was easily made for the rapid

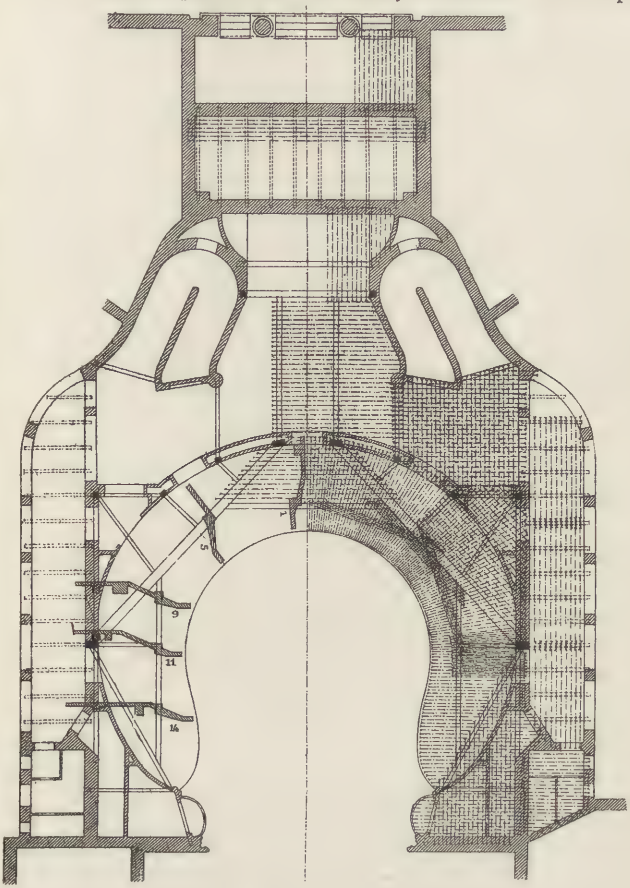


FIG. 264.—Plan of Second Balcony.

emptying of the house by means of exit corridors through the houses on either side. The stage is very little higher

than the lower end of the auditorium floor, which, as usual, slopes down towards the orchestra.

225. Cantilever Construction.—In order to increase the seating accommodation as much as possible the two balconies were made of considerable width, and, so that no obstruction should be caused to the view of those occupying seats below, were designed on the cantilever principle, without any support along the outer edge.

For the realisation of this bold scheme the architect has recourse to the Hennebique ferro-concrete system of construction, which enabled him to utilise to the utmost the limited height of the auditorium, to provide for the support of maximum loads, and to safeguard the building from the risk of fire.

Figs. 263 and 264 are sectional plans which show the general arrangement of the basement, and the crescent-shaped balconies of the auditorium. The chief support of the first balcony is afforded by a beam 12.20 metres long, supported by the division walls of the parterre. The upper balcony is carried by two pairs of beams 11 metres and 7 metres long respectively, placed diagonally, by two beams 9 metres long parallel with the axis of the building, and one beam perpendicular to the same axis, as shown in Fig. 264. These six beams are connected by other members to form a framework for the support of the balcony.

It should here be observed that some of the beams are of very shallow depth, this being necessitated by the limited headroom between the different floors. Thus in view of the fact that the span of the beams carrying the first balcony is 12.20 metres, the depth of these members should have been fully 1 metre, but in order to avoid interference with the view of those who occupy standing room at this part of the theatre the depth was limited to 40 centimetres. This naturally involved the employment of a much larger proportion of steel in the concrete than is usual.

The beams supporting the balcony above were designed under somewhat more favourable conditions. Still, it was necessary to reduce their depth below that which is usual for the spans involved. Typical details of the column and beam construction will be found in Figs. 265 and 266.

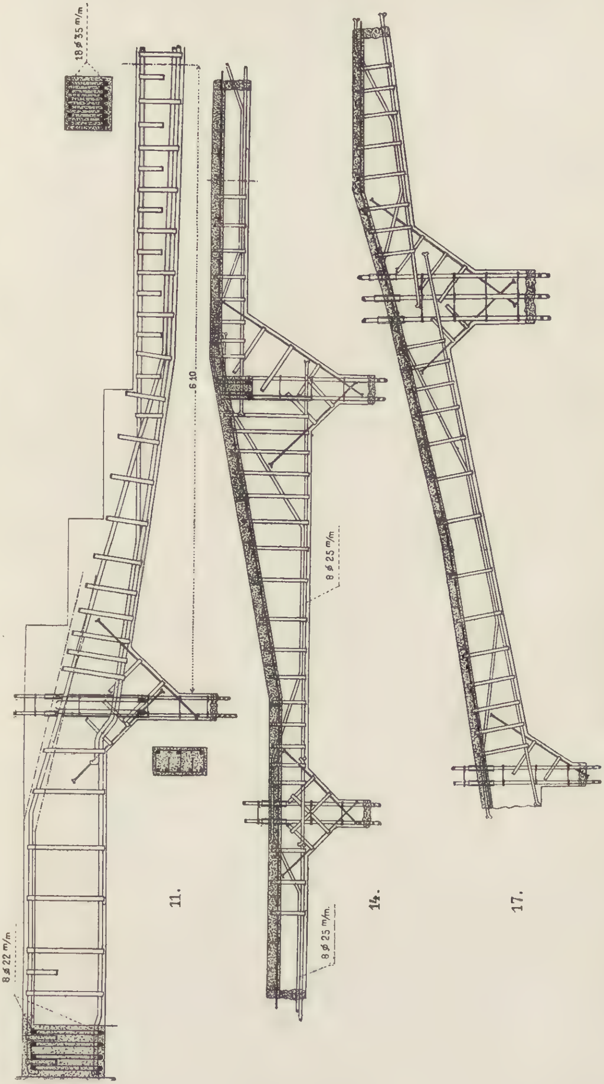


FIG. 265.—Details o Beam and Column Construction.

226. Beam Supports.—The disposition of the beams mentioned had the effect of concentrating weight at several points on the walls of the theatre. As the loads involved

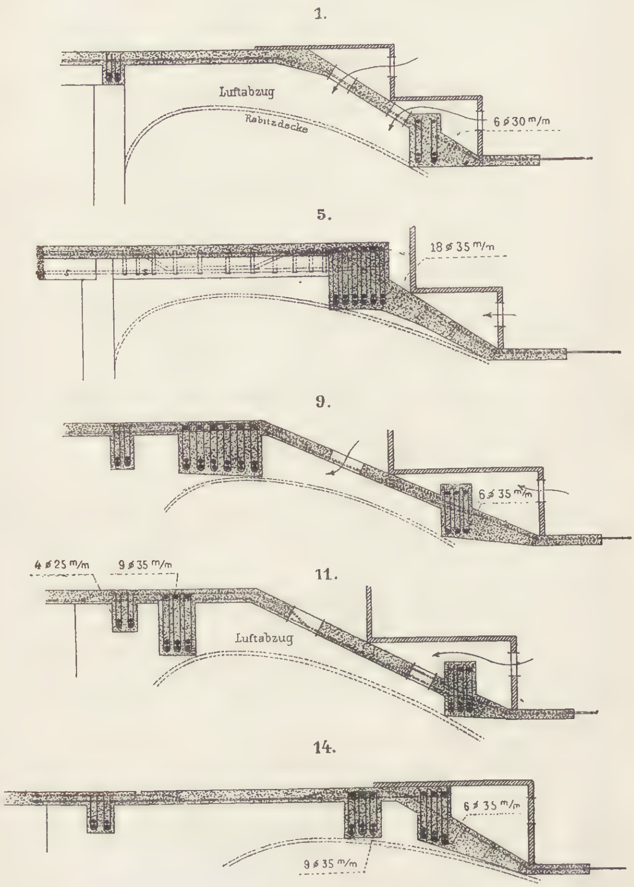


FIG. 266.—Details of Balcony Construction.

greater stresses than the walls were capable of taking with safety, and as the ventilating engineer required numerous openings and chases for ventilation ducts and heating pipes, the architect abandoned the idea of carrying the beam loads upon the walls, and employed columns of concrete-steel, which also serve to support the floor of the parterre. Consequently the walls affected were built in some places so as to be capable of supporting loads, and in others merely as partitions. The columns were continued down into the basement of the building, and supported on concrete-steel footings.

In the upper surface of the balcony floor slabs, strips of timber were introduced during the moulding of the concrete, to which the treads of the steps were afterwards nailed. Outside the auditorium all the horizontal construction, and notably that of the ceilings, roofs, and terraces is in concrete-steel.

227. Ceiling.—The under surface of the balconies is provided with a false ceiling for the double purpose of hiding the projecting ribs and of improving the acoustic properties of the theatre, the space between the two surfaces being useful also for the extraction of vitiated air.

228. Results of Test.—Notwithstanding the minimum beam dimensions adopted, the construction gave extremely satisfactory results when tested in the presence of the municipal authorities. After the lower balcony had been subjected to a load of 800 kilogrammes per square metre —double the calculated superload—the 12.20 metre beam exhibited a deflection of only 1.2 millimetre, or about $\frac{1}{10000}$ of the span. The beams of the upper balcony showed no perceptible deformation after having been subjected to a double load for several days.

One remarkable feature in connection with this building was the short space of time occupied in erection. The foundations were commenced about May 15, 1903; the concrete-steel work was begun by June 10, and finished about September 15 of the same year.

CHAPTER XIII

The Ingalls Building, Cincinnati—Lion Chambers, Glasgow—General Post Office Buildings, London.

229. **Main Features.**—The structure here considered is known as the Ingalls Building, and occupies a site covering an area of 100 ft. by 50 ft. 6 in., at the corner of Vine Street and Fourth Street, Cincinnati, U.S.A. It is built entirely of reinforced concrete, and, although the architectural design is not remarkable for novelty, the structure is worthy of note as the first example of a tall office building in the material mentioned. The general design and the detail drawings were prepared by the architects, Messrs. Elzner & Anderson, of Cincinnati, and the details of the concrete construction—which exemplifies the Ransome system—by the engineer to the Ferro-Concrete Construction Company.

The building comprises sixteen storeys apart from the basement and sub-basement. The distance from floor to floor of the principal storeys is shown in Fig. 267, and it may be added that each storey between the second and fifteenth floors has the uniform height of 12 ft. 6 in. The height of the building from pavement level to the cornice is 210 ft. A particularly noteworthy fact is that the employment of concrete-steel floors permitted a reduction of height equal to 1 ft. for each floor, as compared with the height that would have been necessary if floors with a framework of steel girders had been adopted.

This meant a total saving in the height of 16 ft., and, taking into account the reduced amount of material for walls and interior fittings, it represented a very considerable economy. The cubic measurement saved was $16 \times 100 \times 50.5 = 80{,}800$ cubic ft., which even at the low rate of 6d. per cubic foot is equal to more than £2,000.

230. Wall Foundations.—The building stands upon a stratum of firm gravel and sand, and has extended footings for the walls and columns (see Fig. 268), these footings being situated a little below the level of the basement floors.

After the site had been cleared the walls of the adjoining premises were shored, and in some places underpinned with

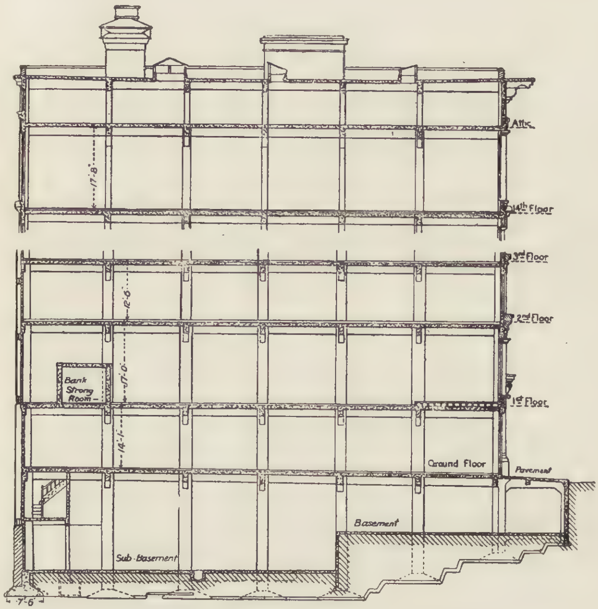


FIG. 267.—Section of the Ingalls Building.

rubble masonry wedged underneath the old walls and thoroughly grouted.

The old party wall on the eastern side of the building—at the top of Fig. 268—had a concrete footing 2 ft. below the present basement floor level, but as the new column bases were to be placed some 6 ft. lower the wall was underpinned and a new footing constructed suitable for the load of 9 tons per linear foot.

The wall at the northern end of the site—at the left hand of the plan—belonged entirely to the owners of the adjoining property, and above the present basement level was carried by columns built into the masonry at intervals of about 16 ft. This wall was supported on needle beams during the removal of the old and the building of the new footings, the latter being situated about 12 ft. lower than the original depth.

Upon the new footings a 21-in. limestone wall was built, surmounted by two 18-in. I-beams for distribution of the

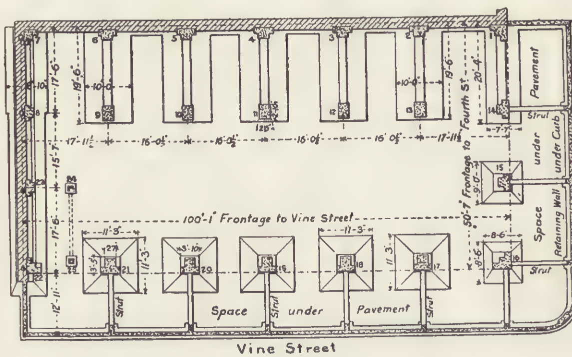


FIG. 268.—Foundation Plan.

column loads. All other loads about the building are transmitted by columns, which are described in Articles 235 and 236.

231. Basement Construction.—For the purpose of permitting the utilisation of space beneath the street pavements, concrete-steel retaining walls were built below the curbs in Fourth and Vine Streets, as indicated in Fig. 268. Along Fourth Street, and as far as column No. 18 in Vine Street, the wall is 14 ft. high by 8 in. thick, and has a footing 2 ft. wide.

The concrete is reinforced by ½-in. vertical and ½-in. continuous horizontal bars, as in Fig. 269, and at intervals by counterforts of concrete, from the upper ends of which

horizontal concrete-steel struts, forming girders for the pavement construction, are carried back to the main columns.

At the corners of the building the retaining walls are bonded by anchor bolts attached to the vertical reinforcement and embedded in the concrete. Between column No. 18 in Vine Street and the north end of the building the retaining wall is 21 ft. high by 24 in. thick at the base, tapering to 8 in. thick at the top. The footing of this wall projects $26\frac{1}{2}$ in. on the outer side and 6 ft. 6 in. on the inner side.

After the construction of the retaining walls they were

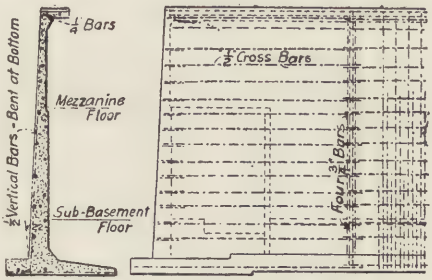


FIG. 269.—Retaining Walls.

supported temporally by shores until the columns and girders had been built.

As the street foot pavement practically forms part of the building it was laid by the contractors. It consists of 16 ft. by 8 ft. concrete-steel slabs $4\frac{5}{8}$ in. thick, and is monolithic with the horizontal girders or struts between the retaining wall and the columns.

Two intersecting series of $\frac{1}{4}$-in. diameter steel bars form the reinforcement of the paving, these bars overlapping 21 in. at all uprights and angles.

The slabs were finished with a layer of mortar consisting of one part of Portland cement and one and a half parts of granite screenings, the upper surface being trowelled smooth and divided into 4-ft. squares by V-shaped grooves filled

with asphalt. Finally, it was sprinkled with granite screenings before the mortar had set.

Below the slabs a 1-in. steel-lath ceiling, constructed in panels, is supported by wire hangers at the level of the lower surface of the girders. The hangers were embedded in the pavement slabs by being passed through holes in the bottom of the moulds.

232. Setting out Columns.—A clear idea of the column spacing will be obtained by inspection of the foundation plan (Fig. 268), wherein all the columns are numbered from 1 to 25.

In setting out the positions of the columns the centres were indicated at ground-floor level by marks on a line near the building line. These marks were laid off by the aid of a transit instrument, and corresponding marks were placed on a line near the top of the adjoining buildings, so that the points could be again laid off in case of interference with the first series of marks.

233. Column Footings.—It will be noticed that nearly all the columns along the street frontages are provided with independent footings, and that the columns in the eastern party wall—at the top of Fig. 268—are supported on footings which also carry the corresponding interior columns. This arrangement was necessary owing to lack of room for the construction of separate foundations along the line of the party wall.

Four of the drawings in Fig. 270 give the details of a typical footing for two columns, as on the eastern wall of the building. In the lower portion or base of this footing there are forty-nine 1-in. twisted steel bars placed transversely, and eight short $\frac{1}{2}$-in. bars placed longitudinally in that part intended to support the larger of the two columns. In the narrow upper portion the reinforcement consists of two $\frac{3}{4}$-in. and ten $1\frac{1}{4}$-in. straight bars, nine $1\frac{1}{4}$-in. twisted steel bars bent as shown in the side elevation, and a series of seven inverted U-shaped stirrups of $\frac{1}{2}$-in. bars. The arrangement of the reinforcing bars is shown more clearly in the enlarged section through AA.

The party wall between the outer columns was formed with them, and, owing to the ample support provided, it

was not necessary to build this wall more than 4 in. thick, except around the windows for the purpose of making suitable provision for fixing the window frames.

Fig. 270 contains the plan and elevation of a typical footing for a single column, as on the western side of the building. The reinforcement is situated near the lower surface of the concrete, and consists of twenty-seven $\frac{3}{4}$-in. bars in one direction and twenty-six $\frac{3}{4}$-in. bars in the other.

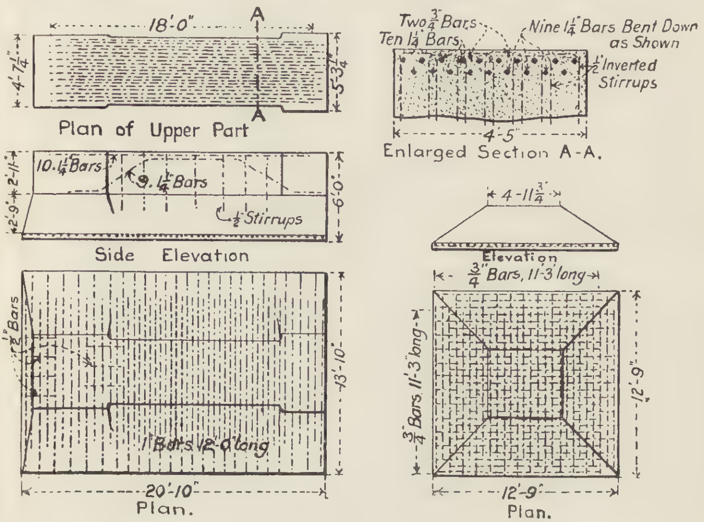


FIG. 270.—Double and Single Column Footings.

234. Column Bases.—In every case the footings as described above were built quite independently of the columns, for each of which a cast-iron base was provided having projecting bosses at the top faced to a true horizontal surface, so as to provide a perfectly level bearing for the faced ends of the vertical reinforcement.

The bases vary in dimensions, but for the purpose of description we will take a base such as was used for columns Nos. 9 to 13 and Nos. 17 to 21.

This, as illustrated in Fig. 271, consists of a base-plate

measuring 4 ft. 6¼ in. square by 3 in. thick at the bottom, from which sixteen ribs—twelve 1½ in. thick and four ⅞ in. thick—rise to the eight projecting bosses, four of 4 in. diameter and four of 3½ in. diameter at the upper part of the base. These ribs are strengthened by horizontal ribs about 1½ in. square cast on the base-plate, one rib carried along the outer sides of the base and the other forming a square in an intermediate position. Four 2-in. holes were cast in the bottom plate to facilitate fixing.

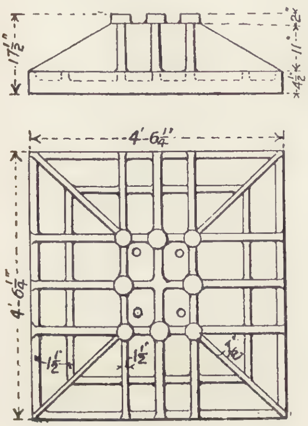


FIG. 271.

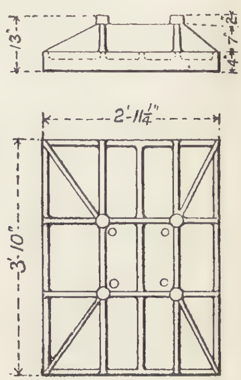


FIG. 272.

Column Bases.

The spaces between the vertical ribs of the bases were purposely left open so that they might be filled up with concrete when the columns were moulded.

Another base as used for columns Nos. 14 and 15 is illustrated in Fig. 272. The general construction is very similar to that above described, but there are only fourteen outer vertical ribs and four bosses for the vertical reinforcement of the columns.

235. Column Details.—The main columns Nos. 1 to 23 measure 38 in. by 34 in. at the base, and are progressively reduced to 12 in. square at the top of the building,

the footings of the different columns varying in dimensions according to the load to be carried.

Fig. 273 includes a part vertical section of a typical column with girder connections, and a cross section of the same column.

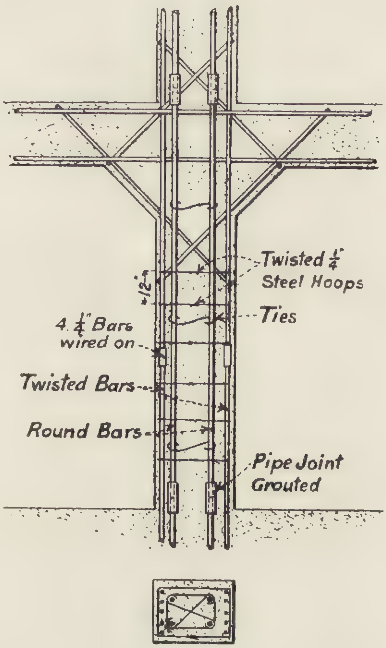


FIG. 273.—Section of a Typical Column.

Each column is provided with four, six, or eight plain round steel bars, ranging from 2 in. to 3 in. diameter, having faced ends held together by sleeves of wrought-iron pipe filled with cement grout.

In the basement these bars take a bearing, as already stated, on the projecting faced bosses of the cast-iron bases.

In the first three storeys the column bars were cut to

length, so that they extended a few inches above the floors, and the next lengths were added as the work proceeded. In the storeys above, the bars were of sufficient length to extend through two floors, the jointing in each case being performed with pipe sleeves and cement grout.

The essential purpose of the round bars is to resist direct compression, but in addition each column has from four to ten twisted bars of square cross section to resist tension due to flexure arising from wind pressure. The column in Fig. 273 has ten twisted bars, placed 1 in. from the surface of the concrete, these bars being joined together at the middle of each storey with splices formed of $\frac{1}{4}$-in. twisted bars wrapped with wire.

In addition to vertical reinforcement the round compression bars are connected by three sets of transverse ties in the height of each storey, to prevent any tendency to outward buckling, and to add to the shearing strength of the columns.

The twisted wind bars are also tied transversely by hoops of $\frac{1}{4}$-in. twisted steel, as shown in Fig. 273. These hoops are spaced at intervals of about 12 in., centre to centre, and secured by wire to the verticals, the ends of each hoop being bound with wire.

The transverse dimensions of the various columns were settled in accordance with architectural requirements, and with the loads to be carried. The resistance of the cross sections so determined were calculated, in the first place, for concrete alone, and the deficiency of strength was then made good by the addition of round steel bars.

Allowance was also made for strain due to shrinking of the concrete, which may cause severe initial stress in the steel reinforcement, and the cross-sectional area of the materials was adjusted so that the stresses might be proportional to the coefficients of elasticity of the concrete and the steel.

236. Column and Girder Connections.—On reference to Fig. 273 it will be seen that very strong girder connections are formed by diagonal bracing bars and concrete brackets on the column at the end of the girders. The diagonal bracing extends from the top of the girder

downward, and from the bottom upward, the lower diagonals being protected by concrete brackets, which not only help to fix the ends of the girders, but also serve as additional wind bracing.

DIAGONAL REINFORCEMENT IN COLUMN CONNECTIONS.

(1)	Two	1-in.	bars	$9\frac{3}{4}$	ft. long.
(2)	„	1	„	$8\frac{1}{4}$	„
(3)	„	$\frac{3}{4}$	„	6	„
(4)	„	$\frac{3}{4}$	„	$5\frac{1}{2}$	„
(5)	„	$\frac{3}{4}$	„	8	„
(6)	„	$\frac{3}{4}$	„	5	„

237. Arrangement of Girders and Beams.—Corresponding with the spacing of the columns, the main girder spans are from 16 ft. to 33 ft. centre to centre, the clear spans, of course, being slightly less. In the ground floor the main girders measure 20 in. wide by 36 in. deep; in the first floor, 20 in. wide by 34 in. deep; and in the three floors above, 20 in. wide by 27 in. deep. The depths stated include the thickness of the floor slabs, which are 7 in. thick in the ground floor and 5 in. thick in all floors above.

Fig. 274 is a typical plan showing the general arrangement of the upper floors; and Fig. 275 is a floor section between the columns from end to end of the building. The widths of the main girders represented in this section are as follow:—

On floors	2 to 4	20 in.
„	5 to 9	18 „
„	10 and 11	17 „
„	12 to attic . . .	16 „

Fig. 276 is a section between the columns across the building.

238. Girder Details.—Fig. 277 gives details of typical girder construction, the most noticeable feature being the apparent preponderance of reinforcement in the upper part of each beam. On closer examination, however, it will be seen that the upper bars only extend for a certain distance from each support. The reason for this is

that owing to the ample strength and rigidity of the column connections the girders were justifiably considered as con-

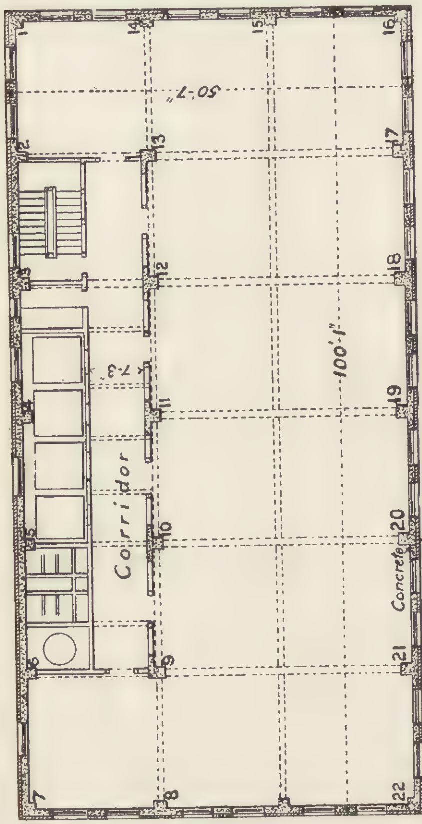


FIG. 274.—Typical Floor Plan.

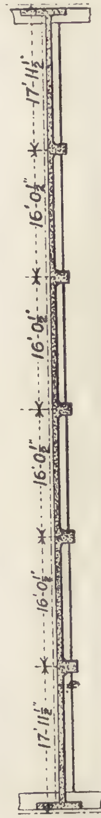


FIG 275.—Typical Longitudinal Floor Section.

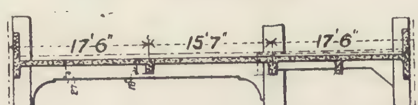


FIG. 276.—Typical Transverse Floor Section.

tinuous beams. Under these circumstances the upper part of the cross section is in tension up to the point of contrary

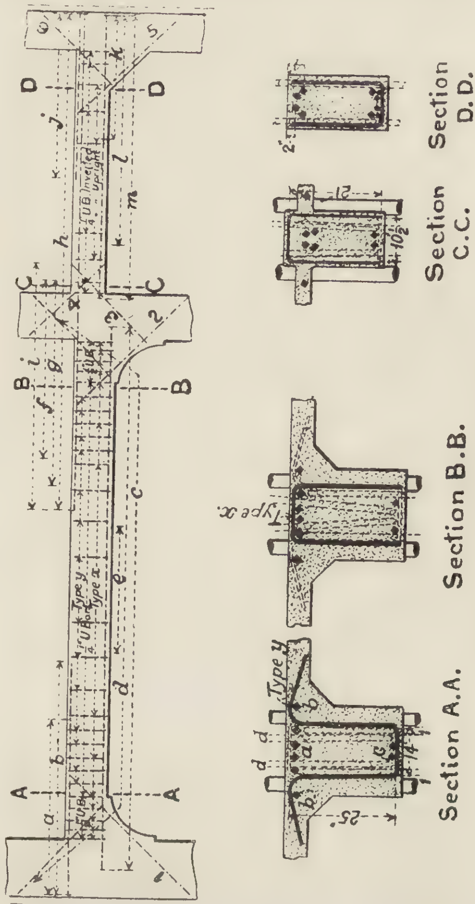


FIG. 277.—Typical Details of Beam Construction.

flexure, a fact accounting for the arrangement of the reinforcement illustrated in Fig. 277.

In addition to the horizontal reinforcement shown, all the girders are amply provided with vertical reinforcement, consisting of U-shaped stirrups of twisted steel bars for resisting shear, every alternate stirrup being inverted. These stirrups are placed at different distances apart, in accordance with the progressive diminution of shearing stress towards the middle of the girder.

The following table gives particulars of the horizontal and diagonal reinforcement in the girders represented in Fig. 277, the letters in this table corresponding with those in the sections :—

HORIZONTAL REINFORCEMENT IN GIRDERS.

(*a*)	Four	$1\frac{1}{4}$-in.	bars	10 ft.	long.
(*b*)	Two	1	,,	13	,,
(*c*)	,,	1	,,	34	,,
(*d*)	,,	1	,,	34	,,
(*e*)	,,	$\frac{1}{2}$	,,	7	,,
(*f*)	,,	$1\frac{1}{4}$	,,	10	,,
(*g*)	,,	$1\frac{1}{4}$	,,	$11\frac{1}{2}$	,,
(*h*)	,,	$1\frac{1}{4}$	,,	34	,,
(*i*)	,,	$1\frac{1}{4}$	,,	14	,,
(*j*)	,,	$1\frac{1}{4}$	,,	$11\frac{1}{2}$	,,
(*k*)	,,	$\frac{3}{4}$	,,	7	,,
(*l*)	,,	1	,,	13	,,
(*m*)	,,	1	,,	16	,,

STIRRUPS IN GIRDERS AT LEFT HAND.

(*x*)	Four	$\frac{1}{4}$-in.	U-bars	5	ft.	4	in.	long.
(*x*)	Ten	$\frac{1}{2}$	,,	5	,,	4		,,
(*y*)	Fourteen	$\frac{1}{4}$	,,	7	,,	8		,,

STIRRUPS IN GIRDER AT RIGHT HAND.

Ten $\frac{1}{4}$-in. U-bars 4 ft. $4\frac{1}{2}$ in. long.

239. Ceilings.—As a general rule the under parts of the floor girders are not covered, but in the corridors of the building a ceiling is formed, the space between the ceilings

and the floor slabs being used as air ducts in connection with the system of ventilation.

240. Floor Slabs.—The floor slabs are reinforced by two series of twisted bars laid longitudinally and transversely, the bars being held in position by the surrounding concrete. Each of the larger floor panels, measuring 32 ft. by 16 ft., is subdivided into two panels, each 16 ft. square, by an intermediate girder (see Fig. 274). Figs. 275 and 276 are typical sections of the floor construction. In calculating the proportions of the reinforcement the panels were treated as slabs, supported along each of the four sides.

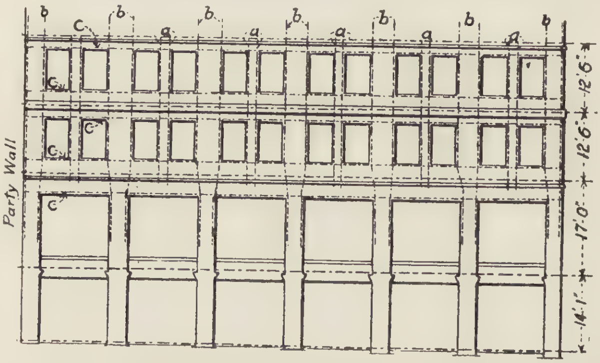


FIG. 278.—Part Elevation and Section of Building.

All the floor slabs are monolithic with the walls, columns, and girders, the calculated live loads being 200 lb. per sq. ft. for the ground floor, 80 lb. per sq. ft. for the first floor, and 60 lb. per sq. ft. for the remainder of the floors.

241. Walls.—Fig. 277 is a part elevation on the Vine Street frontage, from ground level to the fourth floor, and a section of the outer wall. The vertical bars *a* extending through two floors are $\frac{1}{2}$ in. square and 27 ft. long, the ends being jointed close to the floor line. These bars are about 2 in. away from the window openings. The vertical bars *b* form wind reinforcement, the joints being made half-way up each storey. The horizontal reinforcement *c* consists of

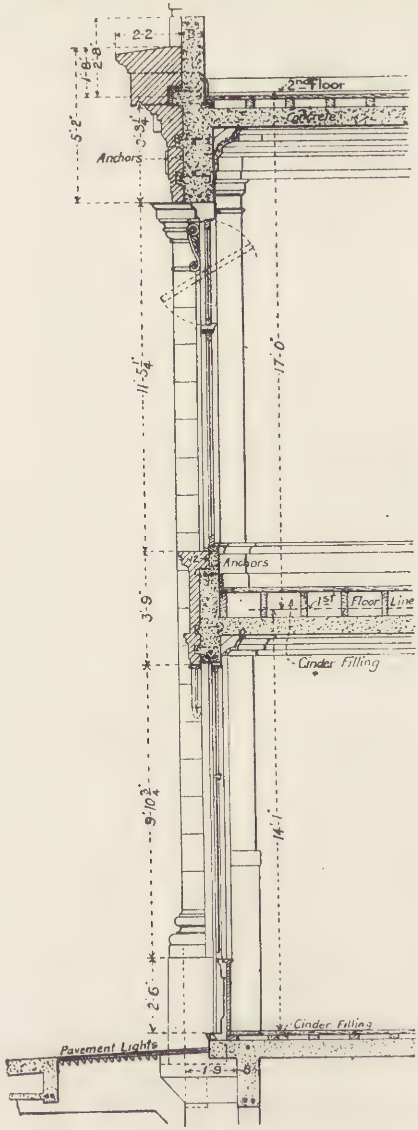


FIG. 279.—Details of Wall Construction.

pairs of ½-in. twisted steel bars set side by side near each face of the wall, and about 1½ in. above the top and bottom edges of the window openings. The bars are 27 ft. long, and their ends are overlapped 21 in. at all joints, the overlapped ends being bound with wire when the joints come between the columns. In the ground floor and first storeys the exterior window and door frames are of cast iron. Although the walls are built throughout of concrete-steel, they are faced externally with 4½-in. marble slabs up to the third floor, and above that the facing is of glazed brick with terra-cotta mouldings. The marble work is attached to the concrete by means of horizontal grooves cut in the back surface of the blocks and engaging projecting ribs formed on the outer surface of the concrete, as shown in Fig. 279. Anchors formed of thin wrought-iron rods are also employed for holding the marble in position, the metal being embedded in the concrete.

The bottom course of the brick facing is laid upon a ledge formed in the concrete, and the brickwork is secured by numerous anchors of the same kind embedded in the concrete.

The terra-cotta used in conjunction with the brick facing is secured by means of grooved joints similar to those adopted for the marble facing.

Irrespective of the marble and brick facing, the outer walls of the building are only 8 in. thick, and the party walls are not more than 4 in. thick.

The concrete of the walls is reinforced throughout with vertical and horizontal bars of twisted steel.

242. **Partitions.**—Partitions dividing up the different storeys into offices and other rooms were built of hollow "Mackolite" blocks, and the ceiling cornices were formed by metal strips, wire netting, and plaster. Apart from doors, windows, and similar interior fittings, these are practically the only portions of the building not constructed of concrete-steel.

243. **Roof.**—As indicated in Fig. 267, the roof is flat, and its construction generally resembles that of the floors. The upper surface of the concrete slab is covered with thick sheets of roofing felt jointed with tar, and over this is a 1½-in.

layer of concrete, faced with neat cement finished smooth and divided into squares by V-shaped grooves filled with asphalt, the same as in the case of the street pavement.

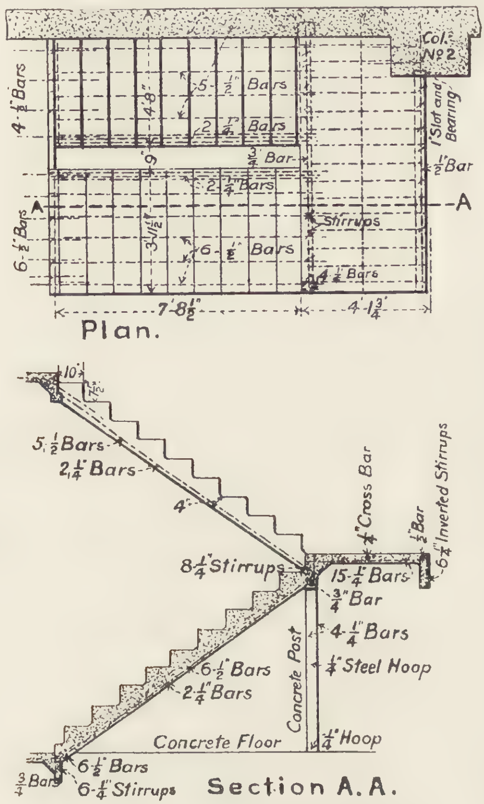


FIG. 280.—Plan and Section of Staircase.

244. Staircases.—Fig. 280 contains a plan and section of a stairway, the position of which is indicated in Fig. 274. As all essential details are clearly shown in the drawings no comment is necessary.

245. Pipe and other Conduits.—Metal sleeves and boxes in the walls and floors for pipes and electric wires were placed in predetermined positions as the work pro-

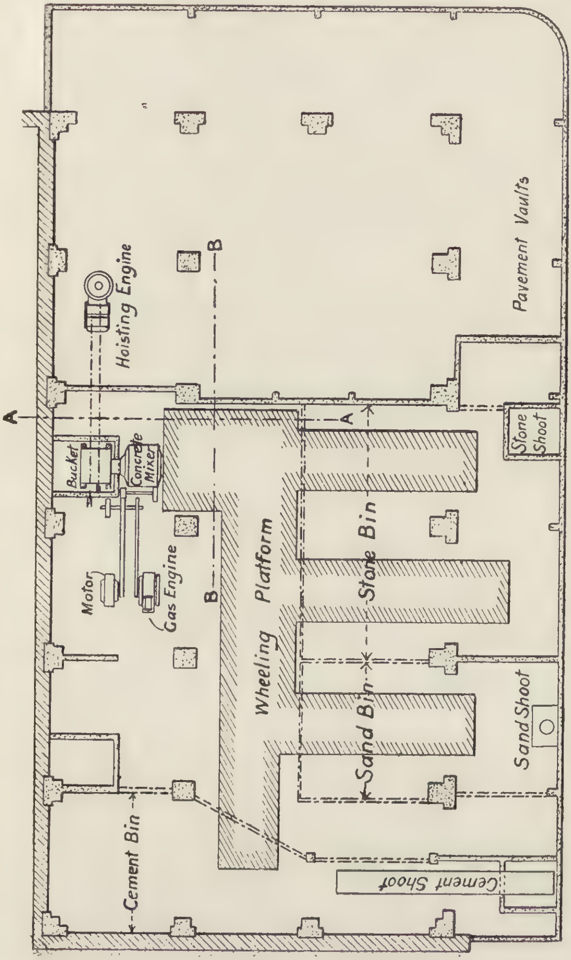


FIG. 281.—Plan of Basement, showing Concrete Plant and Storage Bins.

ceeded, so as to obviate cutting away. To keep out the concrete the sleeves were filled with sand, which was afterwards removed.

246. Storage of Materials.—Full advantage was taken of the basement for the storage of materials, and, as soon as the excavation had been completed three large storerooms (see Fig. 281) were built on the floor for cement, sand, and stone. These were filled from carts or vans by shoots, the positions of which are indicated in Fig. 281, which is a plan of the basement.

As the storerooms contained enough material for the building of two complete storeys, the contractors were fairly secured against the possibility of delays arising from the non-delivery of material at the times expected. A platform was constructed so as to facilitate the wheeling of materials from the storerooms to the concrete-mixing machine, and, to reduce labour, these platforms sloped downwards towards the mixer (Fig. 282).

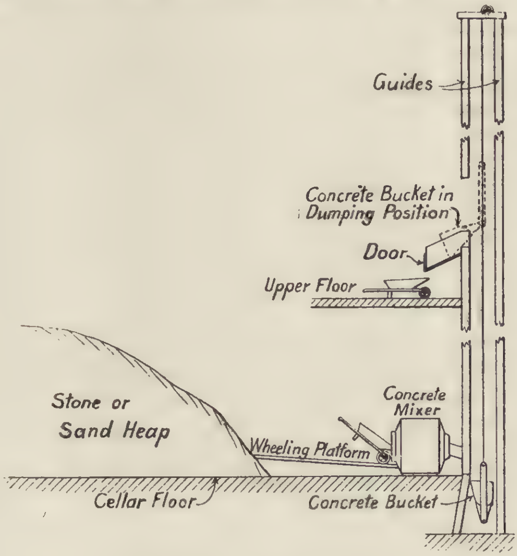
247. Concrete and Hoisting Plant.—At one side of the basement a complete concrete plant was laid down, consisting of a concrete mixer, a concrete hoist, a steam engine and boiler, a gas engine, an electric motor, and the necessary mill gearing (see Figs. 281 and 282).

The concrete mixer was of the revolving drum type, running at ten revolutions per minute, and so arranged that the materials, brought from the storerooms in barrows of known capacity, could be tipped directly into the drum and the mixed concrete afterwards discharged into the bucket of the adjoining hoist without interrupting the revolution of the machine. By this arrangement very little time was lost between the completion of one batch and the commencement of the next. The mixer held about 18 cubic feet of concrete, but as it was not usually possible to utilise the machine to its full capacity the average output was not more than about 10 cub. yds. an hour.

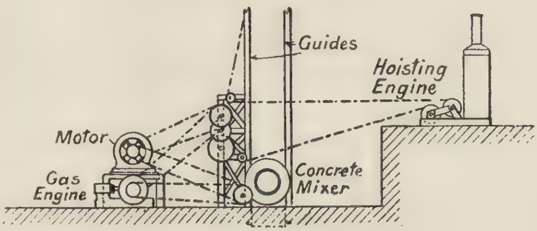
As a general rule the mixer was driven by belting from the electric motor, but to guard against delay owing to temporary breakdown of the motor, provision was made for running it from the gas engine (see Fig. 282).

One of the most noteworthy features in connection with

the construction of this building was the avoidance of undue interference with street traffic, such as occurs when



Elevation on AA (Fig. 281).



Elevation on BB (Fig. 281).

FIG. 282.—Concrete Plant and Hoist.

large steel columns and girders have to be delivered upon the site of a steel-frame building.

It may be mentioned, further, that the large derricks and

other heavy hoisting machinery usually employed in modern building operations were not required, the hoisting apparatus being limited to a light derrick for unloading the steel reinforcement, three or four movable frames for fixing the marble slabs of the lower storeys, and an internal hoist for conveying concrete and timber from the basement to the upper storeys of the building.

The concrete hoist, also available for raising timber, was extended to the upper floors as the building progressed. The hoisting bucket was of special design, being pivoted eccentrically at the bottom and provided with a spout which could be inserted below the outlet of the concrete mixer so as to receive its contents when desired.

When the bucket had been hoisted to any floor it could be tilted so as to discharge concrete into a shoot, provided at the bottom with a simple form of gate valve, whence the concrete was poured out into barrows and wheeled to the required points (see Fig. 282, elevation on AA).

Under ordinary circumstances the hoist was driven by belting from the steam engine, but pulleys and belting were also provided so that it could be operated by the electric motor or the gas engine.

Fig. 282 (elevation on BB) is a diagram showing the alternative methods of driving the mixer and the hoist.

The type of barrow used on the upper floors had two wheels, and was designed so that the concrete could be discharged over the front end, the barrow being wheeled over tracks laid above the tops of the moulds. The concrete mixer, hoist, and barrows were designed and supplied by the Ransome Concrete Manufacturing Company.

248. Proportions of Concrete.—The concrete used throughout the structure consisted of 1 part of Portland cement, 2 parts of clean sharp sand, and 4 parts of pebbles or crushed limestone. A wet mixture was generally used, the proportion of water being varied to suit the different members for which it was required.

249. Bar-Twisting Machine.—Steel bars were twisted on the site by a specially designed machine having three heads revolving at different speeds to suit the sizes of the bars to be treated.

The time required for twisting averaged about one minute, the bars being twisted so that the angle formed by the edge was tangential to an angle of about 20 deg. It is stated that in the case of a 1-in. square bar this effect can be attained by one complete turn in each foot of length.

250. Construction of Moulds.—All moulds for the concrete work were made on the site of the building, the timber being cut as required by a motor-driven saw-bench moved up from floor to floor as the work progressed.

The boards were planed on one face only, to reduce them to the required thickness, the other face being left as it came from the saw so as to give a rough surface to the concrete, with the object of providing a tooth for the plastering.

251. Column Moulds.—The column moulds were strongly built to resist hydrostatic pressure, the sides being formed of vertical boards joined together by being nailed to horizontal battens. The sides of the moulds were held firmly in position by means of horizontal frames built outside them, as will be seen by Fig. 283. Each frame consisted of two stout pieces of timber, placed one on each side of the mould, and connected by bolts passing horizontally through the projecting ends, distance pieces being fixed on the two other sides parallel with the bolts so as to form a rectangular frame. A sufficient number of such frames having been provided for each mould, the whole construction was tightly keyed up by driving timber wedges between the frames and the sides of the moulds.

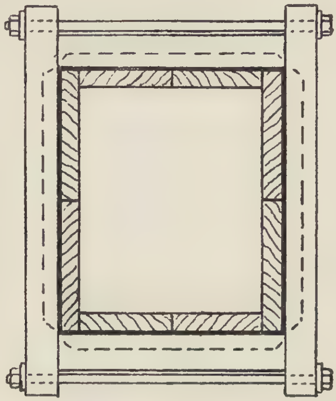


FIG. 283.—Column Mould.

The arrangement differs in point of detail from that illustrated in Fig. 44, but is equivalent so far as concerns its practical effect. In one respect, however, it is more

convenient, as the frame can be very easily removed simply by unscrewing the nuts of the two bolts; then the side boards of the mould come apart quite readily.

After careful adjustment the moulds were secured in position by diagonal braces of timber between each of the four sides and the under surface of the girder moulds.

252. Girder Moulds.—The moulds for the girders (Fig. 284) were of simple construction, consisting of bottom and side boards clamped together, the upper edges of the vertical boards being fixed by nails passing through the planks used for the formation of the floor slabs.

In order to avoid sharp angles at junctions of the girders and floor slabs the girders were sloped outwards at the top, as indicated in the drawing, so as to form a triangular fillet below the floor slabs at the point of junction.

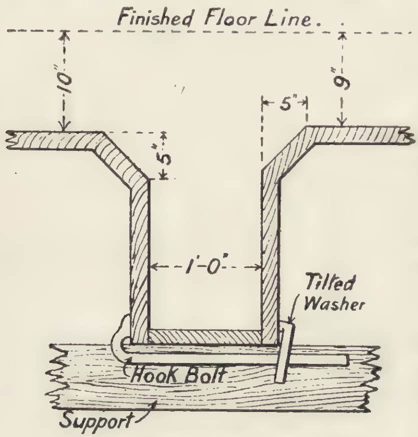


FIG. 284.—Girder Mould.

The clamps employed for holding together the boards at the bottom consisted of a round rod forged at one end into the form of a hook, and provided at the other end with a washer shaped so that when driven to a certain position it would hold the bolt by friction. To tighten the clamp the hooked end is held against the side board of the mould and the washer is then driven forward to the position shown in Fig. 284. To release the clamp it is only necessary to strike a smart blow at the lower part of the washer.

The girder moulds were carried on timber joists attached to the column moulds, and supported by intermediate struts where necessary.

253. Wall Moulds. — Fig. 285 is a vertical section showing part of a mould for the front wall of the building, between the window openings of two storeys. The projecting part of the mould on the left side of the drawing is for the formation of one of the projecting ribs (mentioned in Article 241) by which the marble facing blocks are secured to the concrete. On the other side of the drawing is shown part of the floor centering, the opening above this providing for the junction of the concrete in the floor slab and the wall.

All the wall moulds for parts of the building containing window openings were built up of boards placed horizontally and joined together by outside vertical cleats spaced about 3 ft. apart, and the lower panels were braced outside by timber struts notched into the projecting ends of timbers extending from the girder moulds and scotched below horizontal cleats nailed to the outside of the wall moulds.

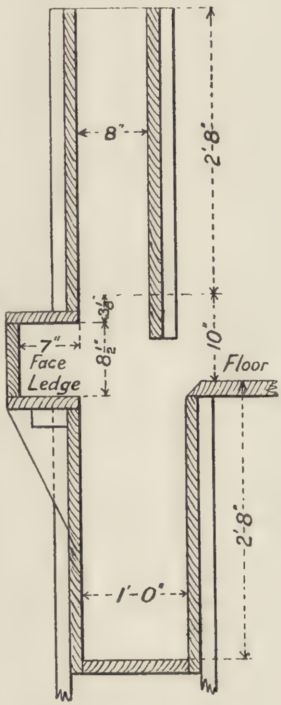


FIG. 285.—Wall Mould.

The wall moulds were supported by struts and bracing to enable them to hold up the semi-fluid concrete, and in the case of moulds next to the party walls the brickwork of the adjoining buildings was utilised, so far as its height permitted, to form the outer side of each mould.

254. Erection of Moulds.—Three sets of moulds were erected at the commencement of work upon the superstructure, and as soon as the bottom storey was completed and the concrete had sufficiently hardened the

corresponding set of moulds was taken down and re-erected for the concreting of the storey above the top set of moulds. This process was repeated until the entire building was completed.

The moulds were kept in position for about fourteen days after the concrete had set, and when they were removed intermediate support was given to the main girders by means of vertical struts for a further period of about thirty days, so as to permit the concrete to attain ample strength before being subjected to the entire load of the floor system.

By proceeding in the way indicated above the contractors were enabled to complete from two and a half to three storeys per month. About ten days were occupied in erecting the moulds for each storey, and two days in depositing the concrete, the quantity being about 120 cub. yds.

255. **Method of Moulding.**—By building the columns first, and following on with the walls, main girders, joists, and floor slabs, the entire weight of the structure was carried by the columns, and, with the exception of the exterior balconies and a few struts, no falseworks were required for the construction of the main features of the building. It may be stated, however, that scaffolding was used by the plasterers and other mechanics in finishing the interior details of the structure.

The concrete was thoroughly stirred and worked in the moulds by means of steel bars or timber poles, so as to ensure the escape of air and the filling of all spaces between the several bars of the reinforcement.

Especial care was taken in the case of narrow girders to fill the moulds slowly, and to stir the concrete at the same time to prevent the formation of voids. In addition to stirring the concrete, the moulds were all struck on the outside with mallets to ensure the settlement of the material and to guard against the appearance of voids on the exterior faces of the work.

256. **Moulding Floors.**—In moulding the floors the reinforcement was first laid, wired in position, and supported on cross rods so as to leave about $\frac{3}{8}$-in. clearance between the bars and the bottom of the mould.

When the concreting had once been commenced it was con-

tinued until the completion of the entire floor, to avoid any break of continuity. After having been deposited the concrete was rammed and the surface dressed smooth and true by straight-edge and level. The girder moulds were filled at the same time as the floor moulds.

257. **Moulding Walls.**—For the convenience of the men engaged in building the wall moulds, in ramming the concrete, and in tapping the outside of the moulds to assist the concrete to fill all spaces, a temporary balcony was constructed round the building at each storey. These balconies were fixed at one end by projecting beams anchored down inside the building, and supported at the free end by diagonal struts bearing against the outer wall.

Tarpauline sheeting was hung below the balcony to catch any drippings from the wet concrete, and thus to protect the marble facing and the men working below.

258. **Staff of Workmen.** — For dealing with the concrete, a staff of twenty-eight men was engaged, made up as follows:—

Wheeling cement, sand, and stone . .	9
Attending to mixer and hoist . . .	1
Attending to hoist on upper floor . .	2
Wheeling concrete on upper floor . .	4
Depositing and ramming concrete on upper floor	12
Total . .	28

Including carpenters and other mechanics, sixty men in all were employed in the execution of the building contract.

LION CHAMBERS, GLASGOW

259. **General Description.**—The office building of which particulars are here given is known as Lion Chambers, and is situated at the corner of Hope Street and Bath Lane, Glasgow. Apart from projecting bays, the structure occupies an area of 46 ft. by 33 ft., and rises to the maximum height of 98 ft. 8 in. above pavement level, or 109 ft. 2 in. from the level of the basement floor.

The building contains eight storeys above the basement, the heights from centre to centre being as follows:—

Basement	10 ft. 6 in.
Ground floor	12 ,, 8 ,,
First ,,	10 ,, 8 ,,
Second ,,	10 ,, 8 ,,
Third ,,	10 ,, 8 ,,
Fourth ,,	10 ,, 4 ,,
Fifth ,,	10 ,, 0 ,,
Sixth ,,	10 ,, 0 ,,
Seventh ,,	11 ,, $8\frac{1}{2}$,,

Figs. 286 and 287 are views taken from different directions in Hope Street, and give a good idea of the architectural design of the building. The ground floor is rectangular in plan, and is divided up into shops and a general entrance hall, from which the main staircase rises to the seven upper storeys and a room in the roof of the corner tower.

The first, second, and third floors are increased in area by the corner cantilever tower, a rectangular projection on the Hope Street façade, and by bay windows on the Bath Lane façade. The fourth floor is further increased by an overhanging projection of the walls supported by corbelling, and the area of the three floors above is still further increased by extension of the outer walls to fill in the spaces between the corner tower and the rectangular projection.

Fig. 288 is a plan showing the arrangement of the sixth and seventh floors, and may be taken as being generally applicable to other storeys.

From foundations to roof concrete-steel has been used exclusively as the material of construction, having been applied substantially on the principle adopted in steel-frame buildings. In other words, the wall columns, interior columns, floor beams, and main roof members were combined to form a complete framework filled in as the work progressed by panels constituting the outer walls, interior partitions, floors, and roof covering, the concrete being monolithic throughout and reinforced by a network of steel bars passing from one part to another and binding the whole system together. Thus from the structural standpoint the

FIG. 286.—Lion Chambers, Glasgow.

FIG. 287.—Lion Chambers, Glasgow.

completed building is virtually a huge rectangular column, with interior stiffening ribs and bracing, firmly fixed at one end into solid earth.

Figs. 289 and 290 are photographic views showing

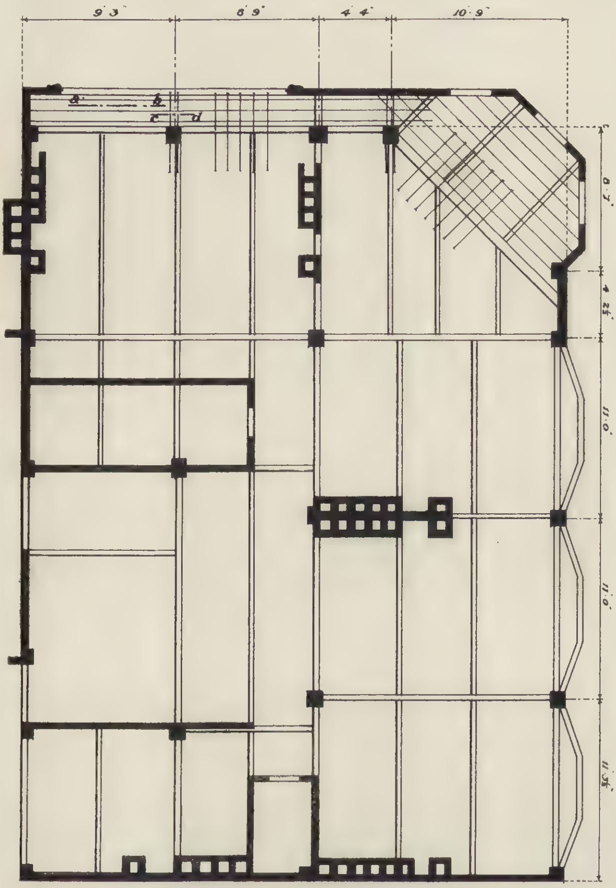


FIG. 288.—Plan of Sixth and Seventh Floors.

respectively the interior of an office on one of the upper floors and an artist's studio under the roof. As indicated by these illustrations, no unnecessary attempt has been made to hide the true features of the construction by meaningless plaster work, as too frequently happens in architectural practice. The work is interesting as the first example in this country of good architectural design realised in a building constructed entirely in concrete-steel.

FIG. 289.—Interior of Office on Upper Floor.

The building was erected from the designs of Messrs. Salmon, Son, & Gillespie, of Glasgow, and the structural drawings of Mr. L. G. Mouchel, M.Soc.C.E. (France), of Westminster, who was represented during execution of the works by Mr. F. A. Macdonald. The contractors were The Yorkshire Hennebique Contracting Company, of Leeds, licensees under the Hennebique patents.

260. Foundations.—All loads constituted by the weight of the building and its contents, by wind pressure, and the weight of snow on the roof, are transmitted to columns

supported on footings below the basement floor, these footings being of sufficient area to keep earth pressure within safe limits.

As the general construction of the foundations is essentially similar to that described and illustrated in previous Articles, detailed particulars need not be given.

It may be mentioned, however, that the foundations are situated at the average depth of 10 ft. below mean pave-

FIG. 290.—View of Studio under Roof.

ment level, thus providing a very secure connection between the whole building and the earth.

261. Columns.—The wall and interior columns, twenty-one in all, extend continuously from the foundations to the roof, being rigidly fixed at the bottom by the footings and the basement floor by monolithic construction and overlapping reinforcement, and fixed in a similar way at each upper floor and at the roof.

Owing to this method of connection the resistance of each column to flexure is double that which would be given by

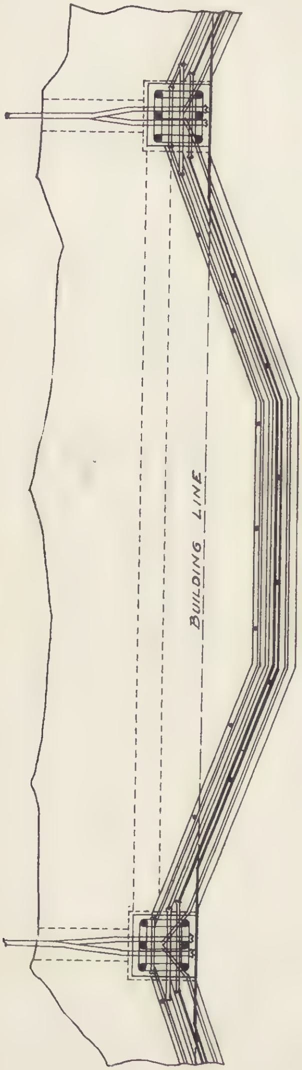


Fig. 291.—Section on line AA (Fig. 294).

a column free to act as if it were round-ended or pivoted, a condition that frequently obtains in steel stanchions as applied in ordinary building construction.

Fig. 288 indicates the positions of the columns, the dimensions of which vary from 13 in. square in the basement to 8 in. square at the top storey, the percentage of the reinforcement being varied conformably with the loads to be sustained. In the case of the wall columns the uniformity of exterior dimensions was governed by architectural considerations, with the incidental advantage that each set of column moulds was suitable for use on every floor. Figs. 291 and 292 contain typical sections illustrating the arrangement of the column reinforcement.

In addition to the main columns, other portions of the building were reinforced suitably for resisting vertical loads, as, for instance, those parts of the outer walls between the windows of the corner tower and the

mullions of some windows in the top storey. These cases are taken in the succeeding Article.

262. Wall Construction.—The sides of the basement are formed by concrete-steel retaining walls 7 in. thick. Fig. 293 is an elevation of the Hope Street façade, and Fig. 294 an elevation of that facing Bath Lane. Read in conjunction with Fig. 288, these drawings show that the spaces between the columns, chiefly filled by windows and in other places by curtain walls which are only 4 in. thick, are carried by wall beams or lintels at each floor and capable of withstanding very heavy loads. The nature of the construction will be made clear by examination of the detail drawings in Figs. 295 to 302.

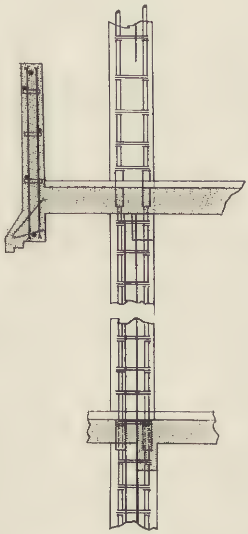


FIG. 292.—Part Section on line CC (Fig. 294).

Fig. 295 is a part section through line GH (Fig. 293), giving particulars of the horizontal and vertical reinforcement at one corner of the rectangular window projecting over Hope Street. Fig. 296 is a part section on line AB (Fig. 293), showing similar details of a bay window.

Fig. 297 is a part section on line GG′ (Fig. 294), illustrating the mullion construction on either side of the window of the corner tower on the side facing Bath Lane.

Fig. 298 is a similar section on the line GG (Fig. 294), representing the wall construction at the same side of the tower.

Fig. 300 is a section on line FF (Fig. 294), further illustrating the wall construction of the tower, and also containing a cross section of the diagonal beams indicated in Fig. 288.

Fig. 301 is a section of the cantilever construction on

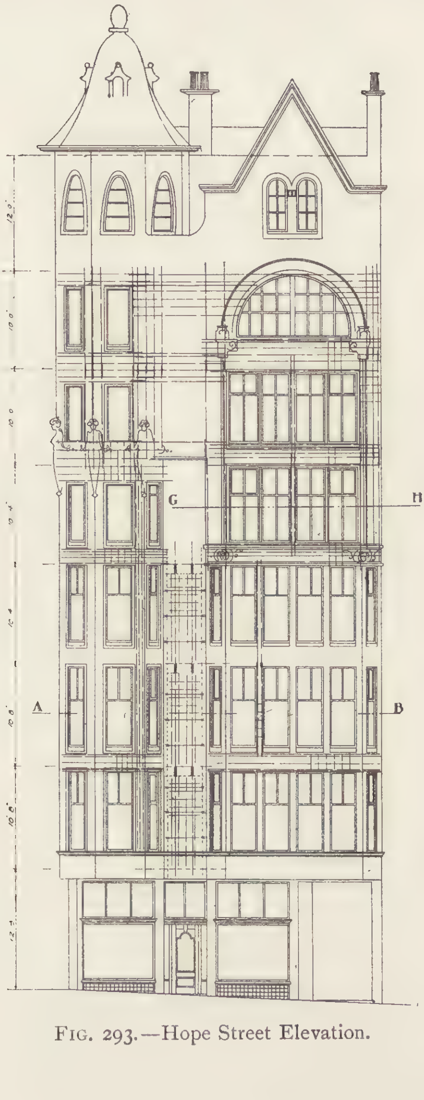


FIG. 293.—Hope Street Elevation.

FIG. 294.—Bath Lane Elevation.

the front façade (see top of Fig. 288), containing part of a main column in elevation, part of a main floor beam with a

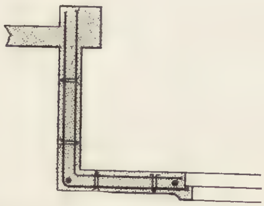


FIG. 295.—Part Section on line GH (Fig. 293).

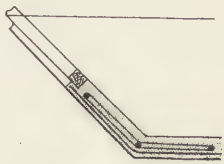


FIG. 296.—Part Section on line AB (Fig. 293).

cantilever projection carrying the rectangular window recess, and part of the front wall.

FIG. 297.—Part Section on line GG′ (Fig. 294).

Fig. 291, p. 344, is a horizontal section on line AA (Fig. 294), between two main columns, representing the wall beneath one of the bay windows facing Bath Lane.

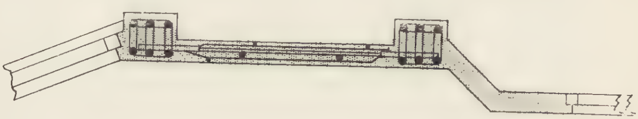


FIG. 298.—Part Section on line GG (Fig. 294).

Fig. 292 is a vertical section on line CC (Fig. 294), showing identical construction in the storey above.

Figs. 299 and 302 give similar details relative to the wall construction on lines EE and DD (Fig. 294).

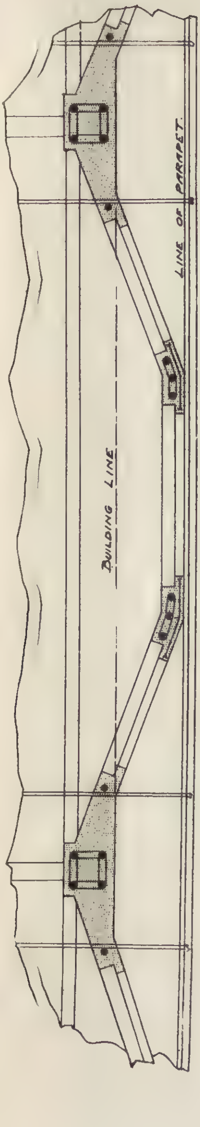


FIG. 299.—Part Section on line EE (Fig. 294).

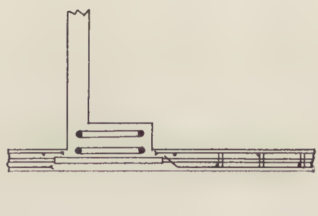


FIG. 300.- Part Section on line FF (Fig. 294).

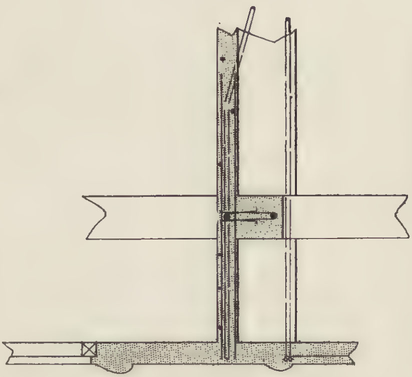


FIG. 301.—Part Section of Front Wall.

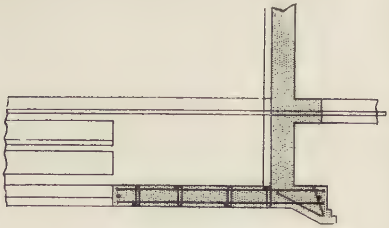


FIG. 302.—Part Section on line DD (Fig. 294).

263. Floor and Roof Construction.—Reference to Fig. 288 will show that the arrangement of the columns and beams is such that the upper floors consist of nine sections, two of three and four panels, respectively, in and near the tower, three of three panels each along the Bath Lane frontage, and four along the party wall, each with four panels of different sizes.

The main floor beams have spans of 15 ft. 1 in. and 17 ft. 10 in. centre to centre, and the spans of the secondary beams, with the exception of those in the tower, have spans of 8 ft. 6 in., 9 ft. 9 in., 11 ft., 11 ft. $3\frac{1}{2}$ in., 12 ft. 10 in., and 15 ft.

The dimensions of the beams are 4 in. to 7 in. wide, and from 6 in. to 12 in. deep below the floor slab, the thickness of the latter on the various floors being as follows:—

Basement	6 in.
Ground floor	4 „
First „	$4\frac{1}{2}$ „
Other floors	4 „

Fig. 303 represents one of the beams in the side wall of the tower facing Bath Lane, and Fig. 304 is a cross section of the cantilever portion of the beam shown in Fig. 301.

Various sections of the floor slab will be found in the illustrations already mentioned. Therefore we need only add two others, Figs. 306 and 307, of the slab in the cantilever projection on the Hope Street façade.

All the floors are designed to carry a superload of 84 lb. per square foot in addition to the dead weight of the construction.

With the exception of the corner tower and the gable on the main façade, the roof of the building is flat, being, in fact, essentially similar to the floors.

Fig. 305 is a section giving details of the roof parapet and slab, and showing also part of a column.

264. Erection.—The space available for use by the contractors during execution of the works was extremely limited.

On the north the site fronts Bath Lane, through which vehicular traffic could not be interrupted, and therefore only

about 3 ft. could be enclosed. On the south the site is bounded by adjoining buildings, on the east by a boundary wall, and on the west it faces Hope Street, where an enclosure of about 6 ft. was permitted.

Owing to the smallness of the area of the site it was

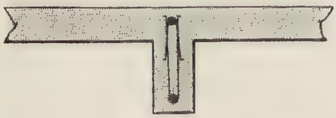


FIG. 303.—Section of Beam in Tower.

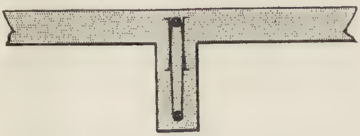


FIG. 304.—Section of Cantilever part of Beam in Fig. 300.

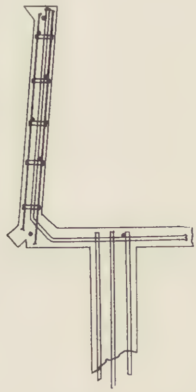


FIG. 305.—Part Section on line HH (Fig. 294).

FIG. 306.—Section *cd* of Floor Slab (Fig. 288).

FIG. 307.—Section *cd* of Floor Slab (Fig. 288).

found impracticable to install power plant for mixing the concrete or for hoisting materials generally.

After the construction had been carried up to street level, part of the ground floor was used as a concrete-mixing shop, whence the material was hoisted externally to the

desired levels. The remainder of the same floor was utilised for the purposes of storage, but being small the accommodation necessitated an almost daily supply of the various materials of construction.

For the reason mentioned above the whole work of erecting this lofty building had to be carried out by hand

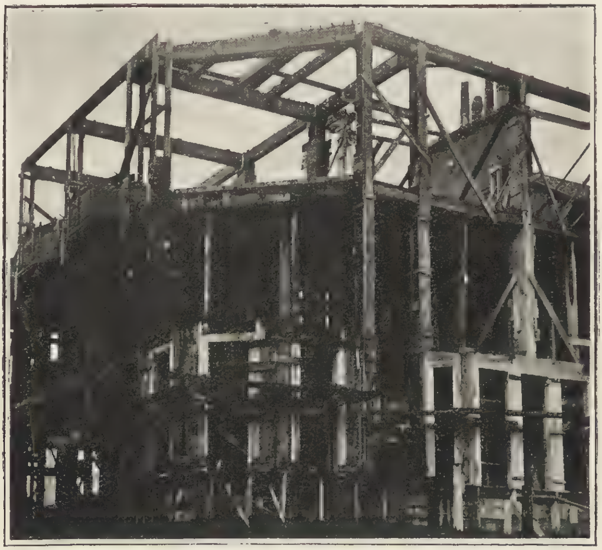


FIG. 308.—View during Construction, showing Moulds and Flying Scaffolds.

labour, all materials being raised by hand winches with the maximum lift of about 90 ft., and outside scaffolding could not be adopted. Flying scaffolds, however, were used as necessary, being supported from each finished floor.

Figs. 308 and 309 are two photographic views illustrating the manner in which external scaffolds were fixed, and the moulds for column, beam, and floor construction.

The method of forming the moulds for concreting requires no special comment, as all the ordinary moulds were of

FIG. 309.—View during Construction, showing Moulds and Flying Scaffolds.

of types generally resembling those illustrated in Figs. 44, 45, and 283.

It may be mentioned, however, that for the cupola of the octagonal tower the timbering was formed by vertical members accurately cut to the contour of the cupola, and between each pair of vertical members short horizontal boards were nailed. The same system was applied to the intrados and the extrados of the work, and the horizontal boards were nailed on as concreting progressed, thus permitting the material to be thoroughly rammed.

Ornamental cornices and mouldings were formed *in situ*, and finished by the plasterer to the exact architectural lines required. Other enrichments, such as medallions, keystones, and figures, representing sculptury—as, for instance, those to be seen at fourth-floor level in Figs. 286 and 287—were cast in advance in strong plaster moulds. When casting these the necessary reinforcement was incorporated with the concrete to enable the details to be efficiently secured and tied to the main structure.

All exterior surfaces were rendered with Portland cement mortar after completion, and great care was taken to secure an even surface, especially on the Hope Street façade.

The exterior surfaces of the cupola and the steep pitched roof surfaces were also finished with a rendering of Portland cement mortar, and for the sake of architectural effect were not covered with asphalt.

The work involved in the construction of the building generally involved much care and minute attention to practical details, and the fact that the serious difficulties attending the erection of so lofty a structure on so confined a site were successfully overcome constitutes one more demonstration of the great adaptability of concrete-steel construction.

GENERAL POST OFFICE BUILDINGS, LONDON

265. General Description.—As a final example of concrete-steel construction we take the extensive buildings, the erection of which on the plot of land formerly occupied

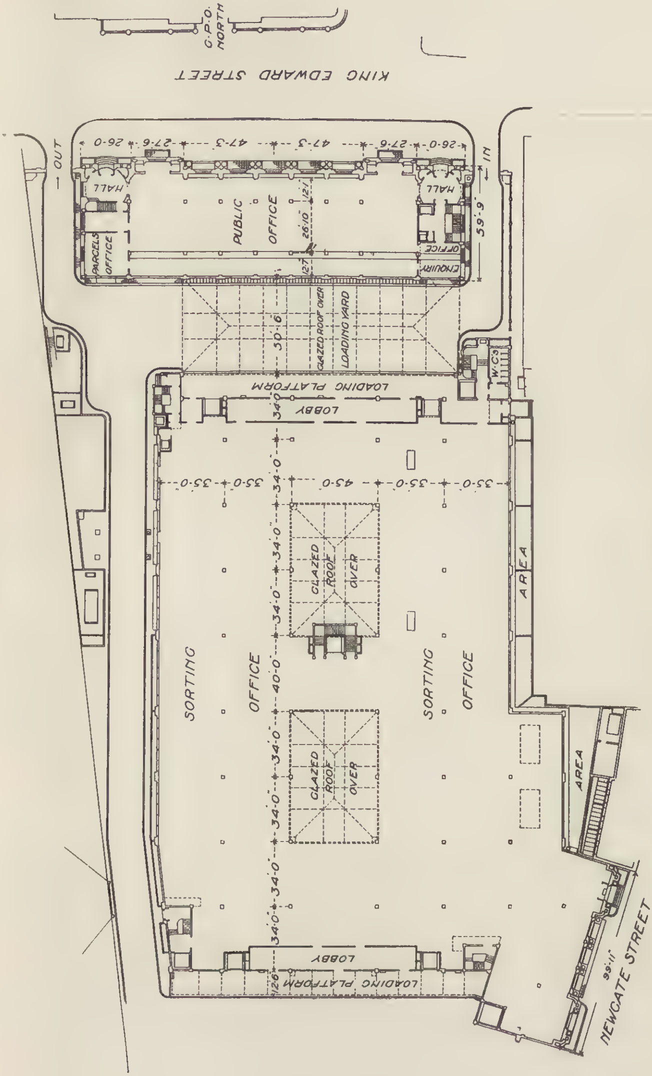


FIG. 310.—New General Post Office Buildings.

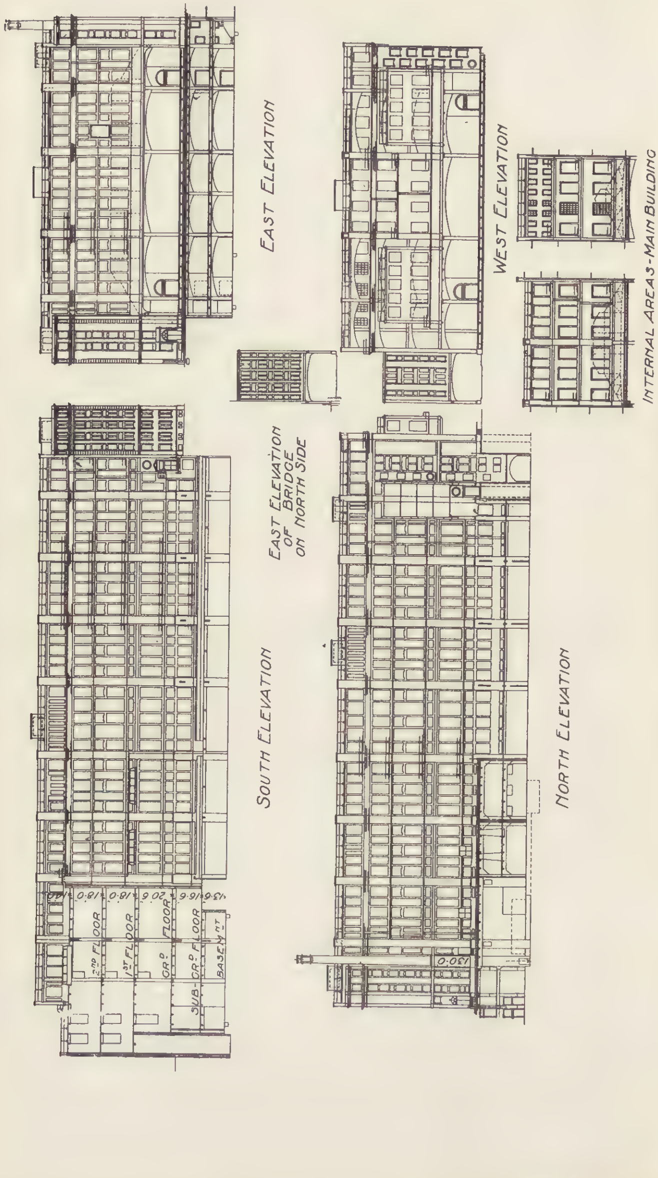


FIG. 311.—New General Post Office Buildings.

by Christ's Hospital has been authorised by H.M. Office of Works as an addition to the General Post Office. They have been designed by Sir Henry Tanner, principal architect to H.M. Office of Works, to whom the author is indebted for the drawings here reproduced as illustrations. All details of the concrete-steel work were prepared by Mr. L. G. Mouchel, M.Soc.C.E. (France), in accordance with the Hennebique system.

Fig. 310 is a ground plan showing the general arrangement of the buildings, which are described officially as the Public Office and Sorting Office. Various elevations and sections are given in Fig. 311. It should be mentioned, however, that the whole of the site beneath and between the two offices is to be occupied by a basement of such extent as to afford accommodation equivalent to that of a third building.

Concrete-steel retaining walls with the average height of 26 ft. are to be built for the purpose of holding up the earth below ground level, and a boiler chimney will rise from the basement to a height of 130 ft.

One very interesting feature is the manner in which the properties of concrete-steel have been applied to cantilever construction on a large scale over the loading platform, where the upper portion of the Sorting Office will project beyond its supports for a distance of nearly 15 ft. The advantage of this arrangement is that an absolutely clear space is left for dealing with mail bags and baskets, an end that could not be attained by steel-framed masonry construction. In the case of concrete-steel the matter is perfectly feasible, for the projecting portion will constitute a huge console connected at every floor to the remainder of the building in such manner as to make ample provision for stability and rigidity.

266. Main Dimensions.—The main dimensions of the Public Office will be 201 ft. long by 60 ft. wide by 85 ft. high above ground level, the total height being 109 ft. It will cover an area of 1,350 square yards, and have a capacity of 1,300,000 cubic ft.

This building will include six storeys in all, the heights of the various storeys being :—

	Ft.	In.
Basement	13	6
Sub-ground floor	15	6
Ground floor	25	0
First floor	14	6
Second floor	14	0
Third floor	14	0
Fourth floor	13	6

The Sorting Office is to be 312 ft. by 185 ft. wide by 63 ft. high above ground level, or 100 ft. high in all. It will cover an area of 6,450 square yards, and have the capacity of 5,780,000 cubic feet. This building is to be founded on a general foundation slab of concrete-steel 5 in. thick, and will include six storeys in all, with the following heights:—

	Ft.	In.
Basement	13	6
Sub-ground floor	16	6
Ground floor	20	6
First floor	18	0
Second floor	18	0
Third floor	14	0

The underground building between the two blocks will include the boiler-house, storerooms, and various offices. It will cover 4,000 square yards of ground and have a capacity of 936,000 cubic ft.

These buildings will probably be completed during the course of the year 1909, and will certainly constitute a most interesting example of concrete-steel construction.

CHAPTER XIV

Some Mishaps and their Lessons

SUBSIDENCE OF GRANARY BUILDINGS IN TUNIS

267. Description of Site and Buildings.—Within recent years the marshy lands bordering the Lake of Tunis, in the vicinity of the city of Tunis, have been drained and utilised in part as the site of the new French quarter commercial port (see Fig. 312).

The buildings to which attention is here directed are—(1) a granary, 56.30 metres long by 14.70 metres wide; (2) a flour mill, 34.50 metres square; and (3) a storehouse, 34.50 metres long by 13.70 metres wide. The height of each may be taken roughly at 24 metres. Fig. 312 shows the position of the buildings, which are represented to larger scale in Fig. 313.

Erected on beds of ancient mud of different densities, and attaining the depth of 30 metres or more, these enormously heavy structures required foundations of exceptional character to ensure stability.

Unfortunately, the designer did not appreciate the extremely unstable nature of the subsoil, and owing to successive movements of the earth all three buildings subsided more or less seriously, as detailed below.

Happily, they were built in concrete-steel, and so sustained no injury. If they had been of ordinary steel or masonry nothing could have saved them from total ruin. As it was, all the buildings were safely restored to the vertical position, and are now as suitable for their purpose as if no disturbance had taken place. However, it remains to be seen whether the foundations will prove themselves capable of withstanding future movements of the earth.

268. Subsidence No. 1.—The first building to subside was the granary. On 22nd April 1906 a gradual movement of the earth commenced, with the result that the granary gradually sank in an outward direction, until it rested at an angle of about 25 degrees with the vertical.

The subsidence is attributed to a depression caused by

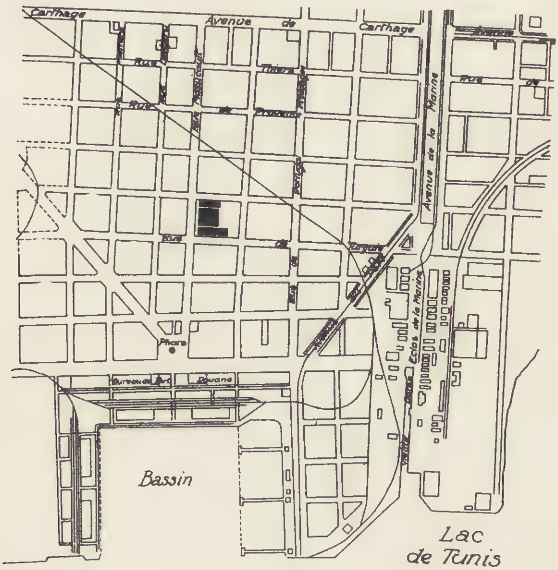


FIG. 312.—Part of New French Quarter, Tunis.

reduction of the water level in the lake and in the water-logged soil around its borders. The outer side of the building rested over the edge of the depression, and not upon earth which, as in the case of that on the other side, was compressed under the weight of the mill.

The condition of the granary after the mishap is thus described by an eye-witness:—

"I have been permitted to enter the building, leaning

like the Tower of Pisa, and have admired the solidity of the construction, which shows no trace of the least fissure and defies disaggregation. Next became manifest the evidence of invincible rigidity in the armoured construction forming the bottom of the silos supported by enormous columns whose original alignment between the two faces of the building had not been disturbed in the slightest degree. It is simply marvellous that such resistance to torsion should have been manifested by so huge a structure, deprived as it were of all support and exposed to the risk of deformations to which it appeared to be doomed by its critical position."

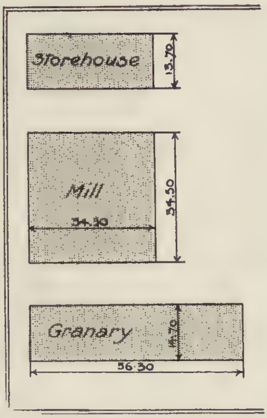


FIG. 313.—Granary Buildings, Tunis, Block Plan.

Notwithstanding the doubts that were expressed as to the possibility of such a course, the building was restored to the vertical position in less than a fortnight by lowering the opposite side of the structure, and if it were not for the fact that the ground floor has now become a basement there would be nothing to show that any disturbance had taken place. The roof structure, shown in Fig. 314, was added to the granary after readjustment, to compensate for loss of the original ground floor.

269. Subsidence No. 2.—Some time after the first subsidence the flour mill was slightly disturbed by a second earth movement. This caused the building to fall over to one side, so that the parapet overhung to the extent 0.50 metre, a mishap that was rectified without serious trouble.

270. Subsidence No. 3.—On the 28th August 1906 a sudden and somewhat extensive movement of the earth occurred on the outer side of the storehouse, causing a double inclination which ultimately caused the parapet to overhang the base by 5.50 metres, one corner of the building being buried 1.50 metres deeper than the other end.

Fig. 314.—Buildings at Tunis, showing the Granary and Mill after restoration to the vertical, and the Storehouse
Fig. 314.—Buildings at Tunis, showing the Granary and Mill after restoration to the vertical, and the Storehouse at maximum inclination.

Fig. 314 is a photographic view, where may be seen the first two buildings after restoration to the vertical, and the third at its maximum angle of inclination.

The mishap to the latter was of far more serious character than the other two. Nevertheless, the building was levelled with perfect success before a month had elapsed, without the least damage to the concrete-steel construction.

271. Method of Levelling.—For the purpose of aiding the restoration of the granary building to a vertical position, some 4,000 tons of sand which has been placed on the various floors, in readiness for load tests about to be conducted, were shifted to the higher side of the building. In addition, exterior platforms were erected, as illustrated in Fig. 315, and loaded with as much stone as could be stacked on them. At the same time, pits were dug along the front (Fig. 315), through which the semi-liquid mud flowed and helped the settlement of the building in the desired direction.

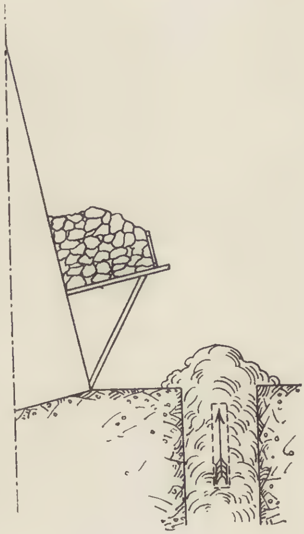


FIG. 315.—Method of Levelling Buildings.

Similar measures were adopted in the case of the two other buildings.

272. Lesson of the Subsidences.—The first lesson of the successive mishaps at Tunis is so obviously the necessity for sound foundation work that it scarcely requires mention. The second is the clearly demonstrated superiority of concrete-steel over any other system of building construction.

For the purpose of enabling the reader to compare the rigidity of these buildings with one of ordinary steel con-

struction we append brief particulars relative to an analogous mishap to a grain elevator in Canada.

The elevator measured 60 ft. wide by 100 ft. high by 180 high, and stood on the bank of the Kamisistikwia River, in Ontario. It was built on concrete foundations supported by 60 ft. piles driven to solid rock. The foundation walls, 16 ft. high, were only 16 in. thick, and far too thin to withstand the enormous load placed upon them. Hence the construction gave way at one corner on the river front and then collapsed entirely, with the result that the building slid bodily for a distance of 30 ft. into the river, where it stood in 20 ft. of water at a considerable angle from the vertical. The structure was of the steel tubular type, and although showing comparatively little sign of damage from the outside was so twisted as to render futile any hope of repair.

A "FLOOR" FAILURE IN YORKSHIRE

273. Particulars of Construction.—The mishap to which we now direct attention is the collapse of a floor built for a small hospital in Yorkshire. The floor was designed to support a uniformly distributed load of about 75 lb. per square ft., in addition to the weight of some brick partitions extending to the ceiling above, and the weight of the floor itself, which consisted of two main beams and a floor slab. The portion of the floor that failed is indicated in Fig. 316. Fig. 317 contains sections further illustrating the construction.

Fig. 318 is a longitudinal section and Fig. 319 a transverse section of a main beam, the transverse section showing part of the floor slab. As there shown, the width of each beam was 17 in., and the projection below the floor slab was 12 in., making the total depth 18 in. The clear span between supports was 25 ft., the ends of the beam being built into brick walls.

The beam reinforcement consisted of two 6-in. by 3-in. rolled steel joists; one $1\frac{1}{2}$-in. "Indented" bar curved upwards to the height of 11 in. above the lower surface of the concrete; and two $1\frac{1}{2}$-in. by $\frac{1}{2}$-in. "Kahn" bars extending for a length of 7 ft. 6 in. on either side of the centre, at a distance

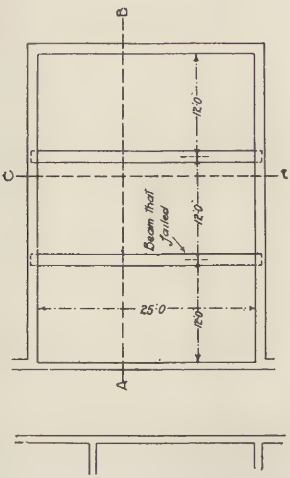


FIG. 316.—Plan of Floor.

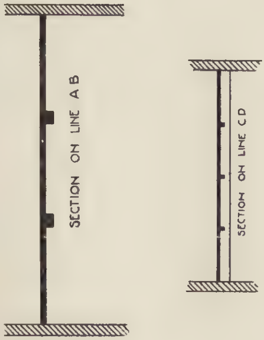


FIG. 317.—Sections of Floor.

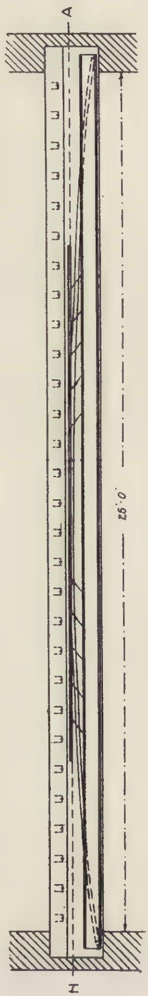


FIG. 318.—Longitudinal Section of Main Beam.

of 7 in. below the upper surface of the concrete, the truss wings of these bars projecting diagonally downwards.

The reinforcement of the floor slab consisted of $1\frac{1}{2}$-in. by $\frac{1}{2}$-in. "Kahn" bars spaced at intervals of 10 in. apart, centre

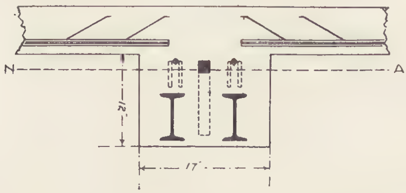


FIG. 319.—Transverse Section of Main Beam.

to centre, the wings of these bars extending diagonally upwards. It should be noted that these bars were not continuous through the beam, as they ought to have been.

274. How Failure occurred.—After the floor had been built in accordance with this scheme, and an interval had elapsed which was thought sufficient to allow the concrete to set, the centring was removed from beneath one beam and the corresponding portion of the slab.

The floor then failed under its own weight and that of the brick partitions above.

275. Discussion of the Design.—Apart from the heterogeneous and disjointed character of the reinforcement, the two joists were of unsuitable section, and the others were placed in positions where their useful qualities could not be properly developed.

The scheme of reinforcement was obviously devised by some one unfamiliar with the principles governing beam design, and particularly with those involved in concrete-steel construction. The cross section suggests the view that the selection of the two rolled steel joists was dictated chiefly by the fact that they would fit conveniently into the space available, rather than by any consideration of their suitability, and that the two patented types of reinforcement were added in the hope that they and a good solid mass of concrete would provide the additional resistance necessary.

Possibly the designer knew something of the principles

underlying the design of beams, but if that were the case this knowledge was only just enough to be dangerous.

For instance, the indented bar shown in Fig. 318 may have been intended to follow the curve suggested by the diagram of bending moments—similar to that in Fig. 320—for a beam with rigidly fixed ends, and the two separate lengths of Kahn bar may have been placed in their isolated positions for the purpose of complying with the supposed requirements of the same diagram.

To a novice the arrangement may look beautifully scientific, but it is at variance with the well-known facts that between the ends and points of contrary flexure (*cc*, Fig. 320) the upper fibres of the beam are in tension and the lower fibres in compression, and that the reverse conditions obtain in the middle length of the beam between the points of contrary flexure. Consequently, the concrete in tension, which always requires reinforcement, gets none from the bars here in question, because they are placed where the concrete is in compression and requires comparatively little aid.

To demonstrate more clearly the mistaken nature of the arrangement, we have indicated in Figs. 318 and 319 the probable position of the neutral axis (NA). It would be impossible to calculate the height of the neutral axis to a nicety without precise information as to the physical properties of the concrete and of the three qualities of steel here used. The position shown has been calculated with average values for the unknown factors, and is sufficiently accurate for the purpose of examining the defects of the beam design.

It is evident that between the points of contrary flexure the curved indented bar is of no use except in helping the concrete to resist compression, and that the assistance so given is almost inappreciable because for the greater part of its length the bar lies near the neutral axis, where neither compression nor tension is developed. For the same reason the two Kahn bars are practically useless, except so far as their projecting wings assist the concrete to resist shear. But as shearing force is at a maximum at each end of the beam, and diminishes to zero at the

centre, the wings are situated where they can not do much good.

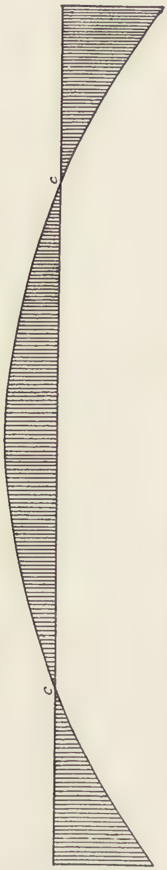


FIG. 320.—Diagram of Bending Moments for Beam with rigidly fixed ends.

Having seen that two elements of the reinforcement are of little or no use as applied in this beam, let us turn next to the two rolled steel joists, which are evidently intended to be the mainstay of the construction. By reference to the cross section in Fig. 319 it will be seen that these members extend nearly to the neutral axis, thereby reducing the capacity of the steel to resist bending moments by shortening the arm of leverage. The result is that the bars are of far less service than is suggested by their imposing appearance.

Without reliable data as to the mechanical properties of the materials used, and particularly to the strength attained by the concrete at the time when the supports were removed, it would be impossible to compute the exact moment of resistance of the beam that failed.

For the purpose of ascertaining the cause of the failure as far as possible, we have calculated the bending moment at the centre of the span, and the moment of resistance of the beam, using average values for the weight and physical properties of the materials.

The bending moment was determined as for a beam simply supported at the ends, because the method of fixing in this case is not adequate for ensuring rigidity to such an extent as would justify treatment of the member as an *encastré* beam. Calculations were made for the dead load only, and for the combined dead load and superload.

The moment of resistance was determined as follows:—(*a*) On the assumption that the concrete was of superior quality and thoroughly hardened, and that the rolled steel joists were of British make, and stressed up to the elastic limit of the metal; (*b*) on the assumption that the concrete was of poor quality and had not fully set, and that the joists were of foreign steel and stressed up to the elastic limit; and (*c*) by employing as factors the values of the maximum stresses permissible for ordinary working conditions, the concrete and the steel being assumed to be of satisfactory quality.

Expressed in terms of the bending moment at the centre of the beam, the results are as tabulated below:—

Moments.	Dead Load only (*A*).	Dead Load (*B*) and Superload.
Bending moment . . .	1.00	1.00
Moment of resistance (*a*) .	0.96	0.66
,, ,, (*b*) .	0.75	0.51
,, ,, (*c*) .	0.24	0.16

In the first column, where the moments of resistance are compared with the bending moment for the dead weight of the construction, line (*a*) suggests that if the materials and workmanship were good the beam must have failed even if the concrete had been allowed to harden sufficiently; line (*b*) indicates that if the concrete and the steel were of indifferent quality the beam was bound to fail under much less than the dead weight alone; and line (*c*) shows that the bending moment due to the dead weight was nearly $2\frac{1}{2}$ times the safe moment of resistance of the beam.

In the second column, where the moments of resistance are compared with the total weight the floor was designed to carry, every line confirms the opinion that the resistance of the construction was utterly inadequate.

The incorrect disposition of the three upper reinforcing bars will be clearly realised by considering Fig. 320 in an inverted position. Then, regarding the beam as one with

fixed ends, the curved line bounding the three shaded areas shows that reinforcement against tension is required near the upper surface at each end, and near the lower surface in the middle of the beam. Further, owing to the heavy load to be carried some reinforcement against compression is needed near the lower surface at each end, and near the upper surface in the middle. The two **I**-beams afford the required assistance at the ends, but there is practically no reinforcement in compression for the upper fibres in the middle of the beam.

In Fig. 321 we have a diagram showing how the varieties of reinforcement already used might have been applied so as to increase the resistance of the construction by nearly 30 per cent. without adding to the weight of reinforcement. The two **I**-beams are left in their original position, which is

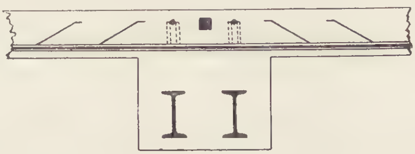


FIG. 321.—Weight of Metal not increased, Original Bars rearranged.

correct as far as such sections permit. The curved bar is straightened out and reduced in area, the metal saved being applied to increase the length of the two short bars, so that they may extend from end to end of the beam. The three bars are placed near the upper surface of the concrete, where they have a greater leverage and are able to act as efficient reinforcement for the concrete in tension at the ends and in compression in the middle of the beam.

Although this revised arrangement offers increased resistance, it would be further improved by substituting ordinary round or other suitable steel bars for the two **I**-beams.

Fig. 322 shows the same quantity of metal as that originally employed, but applied in the form of nine round bars. The upper three bars are for resisting tension at the ends and compression at the middle of the beam; the next three bars are for resisting tension at the ends and

middle of the beam; and the lower three bars are for resisting compression at the ends and tension at the middle of the beam. The result of this rearrangement is to increase the resistance of the beam by 41 per cent.

If the floor had been designed as here shown, with the addition of suitable reinforcement for resisting shear, and if built properly of approved materials and with rigidly fixed ends, it would have been able to withstand the dead load of the construction and the superload also. Still, as the maximum resistance of the beam is still less than the maximum bending moment, for dead load and superload, an increased proportion of reinforcement should be added to ensure stability.

But additional reinforcement is not more necessary than a complete rearrangement of the floor system, which was

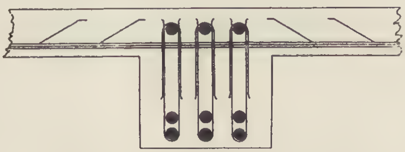


FIG. 322.—Round Bars in place of I-beams (weight of metal not increased).

designed on wrong lines. If the 6-in. floor slab—divided into three large panels by two 17-in. by 12-in. beams—were replaced as suggested in Figs. 323 and 324 by a 4-in. slab divided into twelve panels by 4-in. by 7-in. secondary beams and two 8-in. by 13-in. main beams, the dead load of the floor would be reduced by at least 10,000 lb., with a corresponding diminution of bending moments.

In Figs. 325 and 326 we show a suitable arrangement for the reinforcement of the various members. The floor slab as represented in Fig. 325 is reinforced by steel rods of suitable diameter running in two directions at right angles to each other, and extending continuously over the whole slab, instead of running in one direction only and in disjointed lengths as in Fig. 319. Reinforcement is also added for resisting shearing stresses.

The secondary beams are reinforced, as shown in Figs.

325 and 326, by continuous bars proportioned in accordance with the diagram of bending moments, and bent

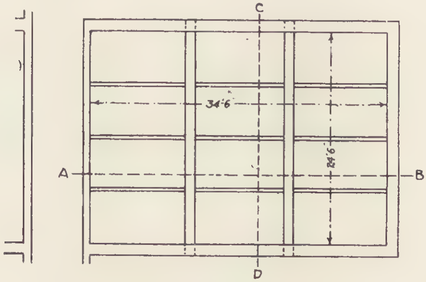


FIG. 323.—Revised Plan of Floor.

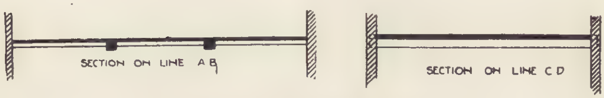


FIG. 324.—Revised Sections of Floor.

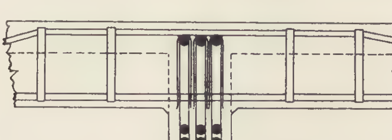


FIG. 325.—Details of Secondary Beam and Floor Slab.

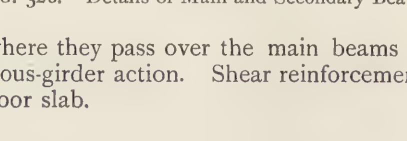


FIG. 326.—Details of Main and Secondary Beam.

upwards, where they pass over the main beams to provide for continuous-girder action. Shear reinforcement is added as in the floor slab.

The main beam is reinforced, as shown in Fig. 326, by three series of longitudinal bars arranged as follows:—(1) The first series extending from end to end near the upper surface of the concrete; (2) the second series commencing at one end immediately under the first series, then dipping down near the point of contrary flexure, continuing along the tension area nearly as far as the other point of contrary flexure, and finally rising to the upper part of the concrete at the other end of the beam; (3) the third series extending from end to end near the lower surface of the concrete. Shear reinforcement is added as in the case of the secondary beams and floors slab.

The application of ordinary steel bars as described above results in an arrangement very similar to that followed in the Hennebique system. If preferred, however, either of the patented forms of reinforcement previously mentioned could be adopted with satisfactory results, providing the bars were suitably proportioned and arranged so as to provide resistance for the calculated stresses.

When a concrete floor is properly designed, and care is taken to secure the monolithic connection of the concrete throughout, it is not a series of separate units but one continuous slab stiffened by projecting ribs and capable of acting as a homogeneous structure. So intimate is the connection between all parts that the slab virtually constitutes a compression flange for all the beams, and the secondary beams act as transverse stiffeners to the main beams. Thus maximum strength and rigidity are obtainable with a minimum expenditure of materials.

By the foregoing discussion it will be seen that the design of a concrete-steel floor involves the careful analysis of stresses, the correct determination of the general proportions of every detail, and reliable calculations relative to the percentage and disposition of the steel used as reinforcement.

276. Lessons of the Failure.—The most conspicuous lesson of this failure is the demonstration that concrete-steel beams and floors cannot be designed by the aid of catalogues alone. The idea that a section book of joists and one or two pamphlets on patented reinforcement constitute a

royal road to concrete-steel design is one that must never be entertained. Rolled steel joists are very useful sections for some work, but are quite out of place in concrete-steel. Indented and Kahn bars are equally useful in the hands of qualified designers, but possess no magic qualities constituting a substitute for knowledge and experience. All who have the necessary theoretical and practical qualifications may safely undertake the design of concrete-steel structures, but others who are not so qualified should abstain from rash experiments, and do nothing without the assistance of an expert.

In view of the probability that the immediate cause of failure was the removal of the centring before the concrete had properly set we may draw the further lesson that the constuction of buildings in concrete-steel ought only to be entrusted to contractors having adequate experience of the new material.

COLLAPSE OF A SWISS HOTEL

277. **Structural Data.**—The building whose failure is here discussed occupied a site at Aeschen, a suburb of Bale, having a frontage of 18 metres and a depth of 46.5 metres. The hotel consisted of two blocks, practically independent, and the mishap occurred to one of these having the width of 18 metres and the depth of 12.5 metres. Fig. 327 is a section illustrating the general design of the building, which it will be seen included a basement and seven storeys. The two side walls and the front wall up to ground-floor level were of brick, this material being also used for the foundations. The upper portion of the front wall was in concrete-steel faced with stone, while the interior columns, column foundations, floors, and some other structural details were also in concrete-steel.

278. **Development of the Failure.**—So far as could be ascertained after the occurrence the collapse was due to the failure of beams 1 and 2 in the first floor (see Fig. 328). These beams were supported by two interior columns and one of the brick party walls.

At the time when the collapse took place the inter-

mediate column was being cased in brick preparatory to the construction of ornamental brick arches beneath the beams. To enable him to carry out this work the contractor removed the centring from the under side of the beams, and did not make provision for supporting them during the operation of casing the column.

As the concrete had not then thoroughly set the beams were unable to withstand the strain coming upon them. Their inevitable failure was followed by the overthrow of the columns and the practical ruin of the building.

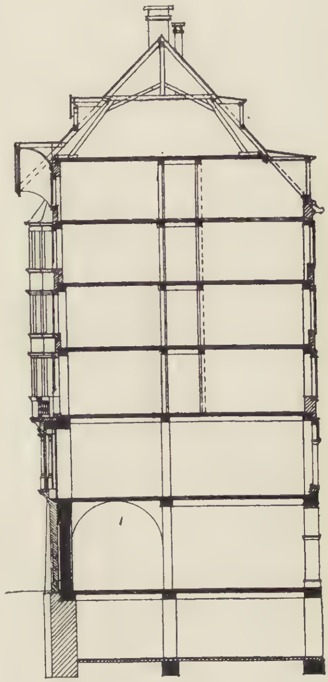


FIG. 327.—Section of Hotel.

279. Report of Experts. — A committee of experts, appointed to investigate the causes of the accident, included Mr. A. Geiser, city architect of Zurich, Professor W. Ritter, and Professor F. Schule. After exhaustive inquiry the committee found that the failure did not indicate any specific fault in the principle of the system of construction adopted, and that the plans and drawings had been satisfactorily prepared. They found also that the removal of the centring on the day of the mishap was contrary to the advice of the engineer representing the designers of the concrete-steel construction, and expressed the opinion that the removal of the supports threw an undue load on the columns before they had completely set. Among the final conclusions in the report of the committee are the following :—

Immediate Cause of the Collapse.—Want of care in removal of the centring from beams 1 and 2 in the first floor, and the absence of support during the casing of the intermediate column.

Contributory Causes.—(1) Inadequate dimensions of the interior column supporting one end of beam 1, and insufficient control by the contractor of the dimensions generally.

(2) The employment of unsuitable aggregate in the concrete.

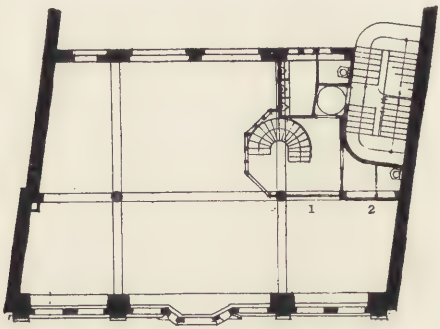


FIG. 328.—Plan of First Floor.

(3) Want of care in executing the concrete work, and particularly inadequate tamping.

(4) Defective organisation of the contractor's staff, with consequent ambiguity as to the authority of the supervising engineer.

(5) Undue haste in execution of the work, and the injudicious removal of centring beneath the various floors.

280. Lessons of the Failure.—From this failure we may learn that, however carefully the designs for a concrete-steel structure may have been prepared, the safety of the construction cannot be secured if executed by a contractor who uses unsuitable materials applied without proper care, and whose foremen insist upon foolish methods of pro-

cedure against the advice of the engineer who has been appointed to supervise the execution of the work.

COLLAPSE OF A BUILDING IN ATLANTIC CITY, U.S.A.

281. Causes of Failure.—From the report of an engineer who examined the ruins of this building, which collapsed during construction, it is clear that the essentials of sound design and workmanship were lacking to a lamentable degree.

The surfaces of the fallen beams and columns were badly split and broken, the concrete was so much injured by frost that large portions could be broken off and crushed between the fingers, and there had never been any proper bond between the concrete of the beams and the floor slabs.

That the necessity for monolithic connection between the parts in question could not have been recognised by the designer was demonstrated by the fact that in several beams the aggregate was trap rock, while in the floor slab broken limestone had been used.

As the designer had evidently included the thickness of the floor slab in the effective depth of the beams, the lack of cohesion necessarily involved a serious departure from the calculated resistance of the construction, and might in itself have been sufficient to account for failure.

Another defect was demonstrated by the shearing of two beams at the supports, thus proving that the reinforcing bars had not been carried into and securely anchored in the supported ends of the beams.

Faults of the kind here indicated are just those likely to occur when concrete-steel work is executed by inexperienced contractors.

PREMATURE FAILURE OF A TEST FLOOR

282. Details of Construction.—The floor here in question was built of hollow bricks and concrete, the reinforcement being applied in the vertical joints between the bricks. The bricks measured 10 in by 4 in., being

laid flat and on edge alternately, with the object of forming a satisfactory bond with the layer of concrete above. The reinforcement consisted of flat steel bars 1.6 in. deep by 0.17 in. wide, and were embedded in cement mortar. The upper layer of concrete had the mean depth of about 7 in., and was mixed in the proportions of Portland cement 1 part, Thames ballast 6 parts; but the specified proportions were 1 part Portland cement to 5 parts Thames ballast. The adoption of the poorer proportions was simply due to neglect of instructions by the builder employed. Moreover, at the time of testing the concrete was still wet, and so had not attained its full strength.

283. Results of Test.—The following particulars are taken from the report of Mr. A. T. Walmisley, M.Inst.C.E., by whom the test was conducted on 31st October 1906:—

At 2.27 p.m. the load of 5 cwt. per square foot was applied, the deflection then being nil; at 2.43 p.m. with the load of 5.96 cwt. per square foot there was still no deflection. Gradual increase of the load up to 7.4 cwt. per square foot caused the deflection of $\frac{3}{20}$ in.; and at 3.50 p.m., when the load of 7.95 cwt. per square foot had been applied, the floor suddenly collapsed. It was then observed that the hollow bricks had failed, and that the tension bars had been pulled out from one end, but were not broken.

In justice to the patentees of the system it should be mentioned that the floor actually fulfilled the conditions for which it was designed, and that if the concrete had been of the proper proportions and thoroughly hardened, the floor would certainly have carried a much greater load than that under which it failed.

284. Lessons of the Test.—This case shows that in incompetent hands concrete may become a dangerous material. The premature failure of the floor is directly attributable to the incorrect proportions, lack of homogeneity, and unseasoned condition of the concrete. It constitutes one more proof of the fact that the average builder's man is quite unfit to deal with concrete-steel construction in any way unless under the most vigilant supervision.

PARTIAL COLLAPSE OF A FACTORY BUILDING IN NEW YORK STATE

285. General Description of Building.—Fig. 329 contains a part plan and two sections of the building which partially collapsed, in November 1906, during erection for the Eastman Kodak Company, in accordance with the Kahn system of tile and reinforced concrete construction.

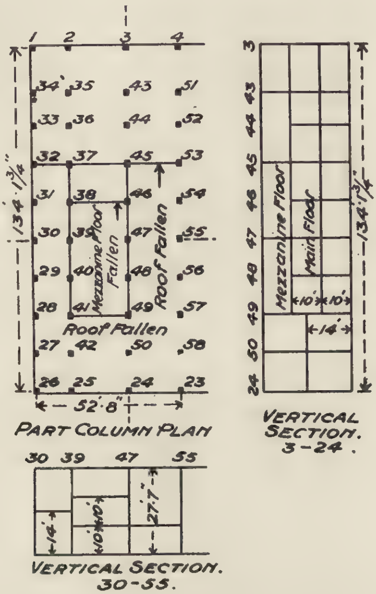


FIG. 329.—Eastman Kodak Building.

The part of the building which failed was 134 ft. long by 53 ft. wide by 28 ft. high, and includes three storeys in part of the area, the remainder having two storeys only. The design was of ordinary character, comprising columns, beams, and slabs. The columns were of concrete reinforced by Kahn bars and some ordinary round bars, the floors and

roof were formed of hollow tiles separated by reinforced concrete beams 4 in. wide reinforced by Kahn bars, and the exterior walls were of brick. At the time of the failure the framework and roof of the building and a considerable proportion of the brickwork had been completed.

286. Extent of the Failure.—The extent of the failure is indicated in the plan (see Fig. 329), the roof and its supporting columns having fallen over an area of about 90 ft. by 32 ft. on the mezzanine floor below, one panel measuring about 45 ft. by 25 ft. was wrecked, but the first floor withstood the shock of the falling materials without further injury than a hole of about 3 ft. square caused by the end of a beam which fell from above.

It is now known that the primary cause of the collapse was the crushing of Column 47, which is stated to have been improperly designed and badly constructed.

287. Report of Expert.—The following facts and illustrations are taken from the report made to the Coroner of Monroe County by Mr. J. Y. M'Clintock, county engineer, by whom the ruins were carefully examined.

Mr. M'Clintock says that in Column 37 at the level of the mezzanine floor there was a horizontal joint of sawdust and shavings nearly through the section, and that in one face of the same column there was an embedded piece of wood about 13 in. long by $5\frac{3}{4}$ in. wide by 2 in. to 3 in. thick. Below the mezzanine floor the concrete was of such quality that at one place a cavity nearly 2 in. deep could be dug into it with a piece of stick.

In many places the steel bars showed at the surface of the beams. For instance, the beams between columns 39 and 40 on floor A had the reinforcement exposed for a length of 6 ft.

In other places the beam reinforcement did not enter the columns, and so there was no proper connection between the members.

Mr. M'Clintock considers the fact that the first floor resisted the impact of falling beams, columns, and roof without appreciable injury as a proof that the general design was satisfactory. He found that the cement and stone used for making the concrete were of good quality, but was led to

the conclusion that dirty sand had been used in places. In many parts of the work the concrete was of bad quality, showing signs of careless preparation and deposition in the moulds, and that the cinder concrete on the roof had the characteristics of earth rather than of concrete.

Mr. M'Clintock adds the remark that "weak spots in the concrete, the honeycombed character of the concrete in the lower part of many columns, and the presence of leaves, sawdust, shavings, and large blocks of wood in the concrete, so as to impair its strength materially, show negligence in construction."

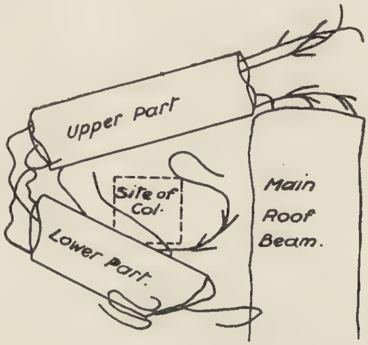


FIG. 330.—Column 47 after failure.

Column 47, to which the failure was primarily due, was indicated in the working drawings with the transverse dimension of 18 in. square to a point 4 ft. above the first floor, and 12 in. square above that point, and with four $\frac{3}{4}$-in. diameter bars from the first floor to the mezzanine floor, and four Kahn bars from the first floor to the roof. These directions were not followed by the contractors, who made the column 12 in. square from the first floor to the roof and altogether omitted the four $\frac{3}{4}$-in. diameter bars.

Fig. 330 is a sketch plan showing the position in which Column 47 was found on examination of the ruins by Mr. M'Clintock. Above the first floor this column was

connected with the main beams of the mezzanine floor and the roof respectively. The concrete in the uppermost part of the column between the roof beams was crushed into fragments so small that they were undiscoverable, and nothing was left but the four Kahn bars. The upper part shown in Fig. 330, about 7 ft. long, was so disintegrated that the concrete near the top could be picked off with the

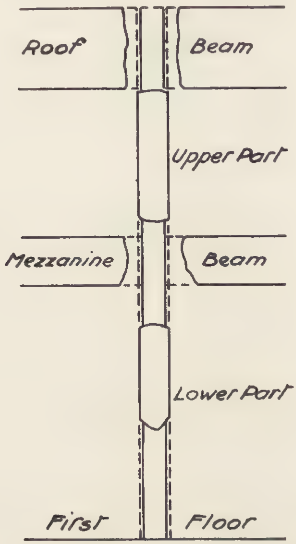


FIG. 331.—Column 47 after restoration to original position.

fingers and crumbled in the hand. The middle part between the mezzanine beams had completely vanished, with the exception of the reinforcement; the lower part indicated in Fig. 330 was found with the outside flaked off in pieces from 6 in. to 10 in. long by from 3 in. to 4 in. wide, and bearing the impress of the reinforcing bars, while the remaining part of the column was broken up into small fragments and the bars were bent into double loops.

Fig. 331 is an elevation showing column 47 as restored to

its original position with the twisted bars straightened out and the main beams propped up in place.

According to the computation made by Mr. M'Clintock, the dead loads on Column 47, including its own weight, were :—

Roof	50,778 lb.
Roof beam	10,500 „
Mezzanine floor	16,227 „
Mezzanine beam	5,190 „
Dead weight of Column . . .	1,950 „
	84,645

As the cross sectional area of the column was 144 sq. in., the dead load was $84,645 \div 144 = 587$ lb. per sq. in. But taking into account the eccentricity of loading due to the fact that the roof slab was 6 in. thick at one side of the column and 10 in. thick at the other, and that the mezzanine floor only occurred at one side of the same member, the equivalent dead load, as estimated by Mr. M'Clintock, was about 950 lb. per sq. in.

Adding to this the live loads, the roof load of 40 lb. per sq. ft. represented by waterproofing materials and snow, and 100 lb. per sq. ft. on the mezzanine floor, we have the further equivalent axial load of 266 lb. per sq. in., giving the total of $950 + 266 = 1,216$ lb. per sq. in.

Each of the four bars used as reinforcement had the sectional area of 0.78 sq. in. or 0.56 sq. in. available as longitudinal reinforcement, the balance being bent outward so as to act as diagonal reinforcement. Therefore the effective area of metal in the four bars would be $4 \times 0.56 = 2.24$ sq. in.

Calculated by the usual formula for short columns of reinforced concrete, the ultimate resistance of the column per square inch would be

$$P = \frac{f_c' \, (a' + a'' \, (m - 1)}{a'}$$

where f_c' = compressive strength of concrete, say, 2000 lbs. per sq. in.
a' = sectional area of the concrete.
a'' = sectional area of the steel.
m = ratio of the coefficients of elasticity of steel and concrete, say, $E'' \div E' = 15$.

Substituting these values, we get

$$P = \frac{2,000[(144 + 2.24(15 - 1)]}{144} = 2,435 \text{ lb. per sq. in.}$$

Consequently the actual factor of safety was practically 2, and even taking into account the four additional bars shown in the drawings, the factor of safety could not be put at more than 2.3. Hence there is no need for wonder that Mr. M'Clintock should have expressed the opinion that, considering the high stresses, the design was imperfect.

In point of fact, owing to the inferior quality of the concrete, its careless deposition, the lack of proper tamping, and the presence of foreign substances, the column was not strong enough to support the dead load alone :—

288. Conclusions of the County Coroner.—In the findings of the Monroe County Coroner the following points are worthy of special note :—

(1) The contractors were found guilty of criminal negligence in submitting working drawings not in accordance with the agreement with the Eastman Kodak Co., and in allowing pressures upon the columns beyond any safe limit.

(2) The contractors' foreman was found guilty of criminal negligence because he allowed the concrete to be put into the moulds without proper tamping, allowed a fewer number of bars to be put in some of the columns than the specified number, and by gross and culpable negligence permitted foreign substances to be built into the concrete of the columns.

(3) The assistant manager of the Eastman Kodak Co., an educated engineer, was found guilty of gross and culpable negligence by permitting the contract to be violated so as to endanger life, by permitting Column 47 to be constructed

far weaker than specified, and by employing a person utterly devoid of building experience as sole inspector without instruction as to the importance of the work he was inspecting, and by not removing work which was obviously defective. He was also found guilty of gross and culpable negligence in permitting Column 47 to be constructed so that it would be subject to a stress considerably greater than is consistent with safety in any building regulations or engineering practice brought to the knowledge of the coroner.

The following conclusions of fact were also drawn by the coroner from the evidence and study of this account :—

(*a*) That the Kahn bars and similar bars with shear members attached are not suitable for columns, because of difficulty in properly tamping the concrete.

(*b*) That no concrete-steel structure can be considered safe unless some reliable man is absolutely responsible for putting the proper number of proper sized bars in the positions designated for them.

(*c*) That the allowable stress upon steel and concrete in construction should be fixed by law, so that the safety of the public shall not depend upon the cupidity of energetic contractors or parsimonious designers.

289. Lessons of the Failure.[1]—So far as can be

[1] Subsequent to the coroner's inquiry the Eastman Kodak Co. have made public a report on the building by Mr. Edwin Thacher and Mr. C. W. Marx, who were commissioned by the company to investigate the strength of the design and the safety of the construction. These engineers say: "We are fully convinced that the primary cause of the failure was due to the fact that the supports under girders and floor construction were removed too soon for that season of the year, your records showing that the concrete was only about three weeks old. The concrete throughout is of good quality, but was too green to enable the respective members to even carry their respective dead loads with safety." Messrs. Thacher and Marx disagree with the estimates of dead load made by Mr. M'Clintock, and calculating on the basis of 3,000 lb. per sq. in. as the ultimate strength of the concrete, arrive at the result that Column 47 as built was not improperly designed. The assumed value of 3,000 lb. per sq. in. is clearly excessive for ordinary concrete, especially when applied in columns. It is unfortunate that instructions were not given at an earlier date, so that this report might have been considered together with that of the county engineer at the official inquiry.

judged by the evidence published, a good deal of the "criminal negligence" and "gross and culpable negligence" displayed in connection with this disastrous failure was due to the fact that an independent engineer was not employed to design and superintend the execution of the works.

The working drawings were furnished by the contractors, and the superintendence of the works was delegated to a perfectly ignorant and incompetent person. To accept a basis such as this is simply to court disaster.

One important lesson to be learned from the occurrence is that patented forms of reinforcement, however good they may be when correctly applied to members for which they are suitable, do not provide a substitute for knowledge, skill, and experience, nor for that careful attention to all details of design and that thorough and conscientious oversight over construction which is only to be secured by obtaining the co-operation of a responsible and independent engineer.

APPENDIX

LISTS OF CONCRETE-STEEL BUILDINGS AND OTHER STRUCTURES IN THE UNITED KINGDOM

In compiling the subjoined lists the Author has been influenced by the hope that the information there presented may be useful to readers who desire to inspect actual examples of concrete-steel construction.

It should be mentioned that floors are not included except where incidental to structures fairly coming within the category of concrete-steel buildings. There are two reasons for this. One is that the inclusion of floors would have made the lists inordinately long. The other is that it is almost impossible to draw a clear dividing line between "concrete-steel" floors and "steel and concrete" floors. Consequently, any attempt to differentiate between the two classes must have led to invidious distinctions in the case of some types occupying intermediate positions.

No indication is given as to the systems represented by the structures in the various lists, because most of the engineers or firms interested in this special class of work have only commenced operations within a comparatively recent period, and it would scarcely be fair to emphasise the numerical inferiority of the works they have executed as compared with the far larger number completed by the pioneers of concrete-steel construction in this country.

BRIDGES.

Birmingham	Accommodation Bridge	City Council.
,,	Saltley Road Viaduct	City Council.
,,	Fazeley Street Bridge	City Council.
,, (Smethwick)	Highway Bridge	Borough Council.
,, (Stirchley)	Bond Street Bridge	District Council.
,, ,,	Umberslade Road Bridge	District Council.
Bournemouth	Tuckton Bridge	Borough Council.
Bristol	River Frome Covering	City Council.
Carlow	Decking	County Council.
Chewton (Hampshire)	Skew Arch Bridge	Mr. H. J. Weston.
Clitheroe	Knowlmere Manor Bridge	Mr. William Peel.
Colne	Highway Bridge	Borough Council.
,,	River Covering	Borough Council.
Crewe	Crewe Hall Bridge	The Earl of Crew.
Derby	Milford Bridge (widening)	County Council.
Dundee	Bridge over Slipway	Dundee Harbour Trust.
Hamilton (Cadzow)	Decking	Borough Council.
Hull	Highway Bridge	City Council.
Lamesley (Durham)	Highway Bridge	Parish Council.
Leeds	Decking	City Council.
Leigh (Lancs.)	Butts Bridge	Borough Council.
Lilburn	Highway Bridge	
Liverpool	Burlington Street Bridge	City Council.
London (Denmark Hill)	Decking	County Council.
,, (Weybridge)	Motor Track Bridge	Brookland Automobile Racing Track Co.
Lucker	Highway Bridge	
Manchester (Astley)	Canal Bridge	Manchester Ship Canal Co.
Middleton	River Covering	Messrs. Mather, Ormshaw & Co.
Nelson	Highway Bridge	Borough Council.
Northallerton	Crambeck Bridge	North Riding County Council.
Reedsmouth	Decking	District Council.
Rochdale	Mellor Street Bridges	Borough Council.
,,	Firgrove Bridge	Borough Council.
,,	River Roch Covering	Borough Council.
Rugby	Footbridge, River Avon	

Skipton . . .	Highway Bridge . .	District Council.
Southampton . .	Blackwater Bridge. .	Borough Council.
Stainburn . .	Highway Bridge . .	West Riding County Council.
Stamford Bridge .	Highway Bridge . .	County Council.
Ulverston (Satterthwaite)	Highway Bridge . .	County Council.
Waterford . .	Knockruabon Bridge .	County Council.

BUILDINGS.

Accrington (Church).	Mill Extensions . .	Messrs. F. Steiner & Co.
Avonmouth . .	Sheds and Granary .	Bristol Docks Committee.
Belfast . . .	G.F.S. Lodge . .	D.M. of D. and C.
,, . . .	Dyeing and Cleaning Works	Monarch Laundry Co.
,, . . .	Stabling and Storage .	Messrs. Wallis & Co.
,, . . .	Linen Factory . .	Messrs. Somerset & Co.
Birkenhead . .	Granary . . .	Mersey Docks and Harbour Board.
Birmingham . .	Wolseley Works . .	Wolseley Motor Car Co.
,, (Aston) .	Stellite Works . .	The Electrical Ordnance and Accessories Co.
Bournemouth . .	Elementary School .	Borough Council.
,, (Queen's Park)	Golf Pavilion . .	Borough Council.
Bolton . . .	Warehouse . . .	Messrs. Joshua Barber & Co.
Bradford . . .	Waterloo Mills . .	Mr. J. Reddihough.
,, . . .	Offices	Tramway Co.
,, . . .	New Zealand Warehouse	Mr. James Hill.
,, . . .	Quebec Warehouse .	John Smith's Trust.
Brentford . . .	Warehouse . . .	G.W.R. Co.
Bristol (Canon's Marsh)	Goods Station . .	G.W.R. Co.
,, ,,	Transport Sheds . .	Docks Committee.
,, . . .	Electric Transformer Station	City Council.
Cardiff . . .	Coal Washery . .	Cardiff Washed Coal Co.
,, . . .	Library and Offices .	Engineering Institute.
,, . . .	Business Premises, Queen Street . . .	Messrs. J. Williams & Co.
,, . . .	Warehouse and Offices .	G.W.R. Co.
,, . . .	Granary . . .	Messrs. Noah, Rees & Co.

Carlisle . . .	Factory	Messrs. Hudson, Scott & Sons.
Carmarthen . .	Warehouse . . .	Western Counties Agricultural Co-operative Association.
Chandlers Ford .	Pumping Station . .	South Hants Water Co.
Chatham . . .	Volunteer Drill Hall and Headquarters . .	Royal West Kent Regiment.
,, . . .	School of Electricity .	War Office.
Colne . . .	Warehouse . . .	Colne Co-operative Society.
Dublin . . .	Bonded Warehouse .	Messrs. J. Jameson & Son.
,, . . .	Printing Works . .	Messrs. Hely & Co.
Dundee . . .	Bonded Warehouse .	Messrs. J. Watson.
Edinburgh . .	Strong Room . .	Mr. A. Hunter Crawford.
,, . . .	Paper Stores . . .	Messrs. T. Nelson & Sons.
,, . . .	Printing Works . .	Messrs. T. Nelson & Sons.
,, (Leith) .	Biscuit Factory . .	Messrs. Wm. Crawford & Son.
Glasgow (Hope St.).	Office Building . .	Mr. W. G. Black.
,, (Polmadie).	Pattern Shop. . .	Messrs. Alley & MacLellan.
Gloucester . .	Offices	Messrs. Price, Walker & Co.
,, . . .	Derby Road Council Schools	Borough Council.
Handsworth . .	Stables	District Council.
Harrow . . .	Factory. . . .	Messrs. Kodak.
Hartlepool, West .	Shop (Lynn Street) .	Messrs. Robinson & Co.
Harwich . . .	Goods Shed . . .	G.E.R. Co.
Horsforth . .	Sewage Works . .	District Council.
Hull . . .	Foundry, Workshop, and Offices	Messrs. Rose, Downs & Thompson.
,, . . .	Factory. . . .	Messrs. Blundell, Spence & Co.
Ipswich . . .	Granaries and Grain Cleaning House	Messrs. R. & W. Paul.
Leeds . . .	Premises . . .	Y.M.C.A.
,, . . .	Motor Garage . .	Yorkshire Mutual Garage Co.
,, . . .	Sunday School . .	Corporation.
Leigh . . .	Brewery . . .	Messrs. G. Shaw & Co.
Liverpool . .	Premises . . .	Messrs. Evans, Sons, Lescher & Webb.

Liverpool . .	Albion Oil Mill, Boundary Street . . .	Messrs. Simonds, Hunt & Montgomery.
,, . . .	Sandon Dock Sheds .	Mersey Docks and Harbour Board.
London . . .	Warehouse . . .	The Tower Tea Co.
,, . . .	Warehouse . . .	London and East India Dock Co.
,, (Acton) . .	Printing Works . .	Messrs. Röder & Co.
,, (Battersea) .	Warehouses . . .	Messrs. Hampton & Sons.
,, (Cheapside) .	Offices	Scottish Temperance Life Assurance.
,, (City) . .	Strong Rooms . .	Basildon House, Moorgate Street.
,, (Grosvenor Sq.)	Residence (rebuilding) .	
,, (Hatfield St.) .	Printing Works . .	Messrs. Hudson & Kearns.
,, (Halton Gdn.).	Strong Room and Partition Walls	Mr. Stewart Dawson.
,, (Nag's Head, Borough)	Warehouse . . .	G.W.R. Co.
,, (Pall Mall) .	Bankers' Strong Room .	Messrs. Brown, Shipley & Co.
,, (Pimlico) .	Warehouse . . .	Associated Portland Cement Manufacturers.
,, (Royal Albert Dock)	Warehouse . . .	G.W.R. Co.
,, (Royal Oak) .	Stationery Warehouse .	G.W.R. Co.
,, (St. James's Sq.)	Offices	Clerical and Medical Life Insurance Co.
,, (St. James's St.)	Strong Room . .	London and County Banking Co.
,, (St. Martins le Grand)	General Post Office Extension	H.M. Office of Works.
,, (Shepherd's Bush)	Cooling Tower . .	Central London Railway.
,, (Silvertown) .	Warehouse . . .	Messrs. J. Knight & Son.
,, (Tavistock St.)	Strong Room . .	Sir George Newnes.
,, (Tottenham) .	Storehouse . . .	Messrs. Whitbread & Co.
,, (Victoria Dock)	Flour Mills and Granary	Messrs. Wm. Vernon & Sons.
,, (Willesden) .	Storehouse . . .	Messrs. Whitbread & Co.
,, (Woolwich) .	Submarine Stores . .	War Office.
Manchester . .	Central Offices . .	Co-operative Wholesale Society.
,, . . .	Warehouse . . .	Co-operative Wholesale Society.
,, . . .	Dock Warehouses . .	Manchester Ship Canal Co.
Middleton . .	Refuse Destructor . .	Borough Council.

Newcastle-on-Tyne	Offices, Forth Bank	N.E.R. Co.
,,	Benwell Refuse Destructor	City Council.
,,	Granary	Messrs. Spiller & Baker.
,,	County Hall	County Council.
,,	Forth Bank Warehouse	N.E.R. Co.
,,	Warehouse	Co-operative Wholesale Society.
,, (Dunston)	Granaries (two) and Grain Cleaning House	Co-operative Wholesale Society.
,, (Jarrow)	Store and Shops	Jarrow and Hepburn Co-operative Society.
,, (New Bridge Street)	Goods Station	N.E.R. Co.
,, (North Shields)	Warehouse	North Shields Fish Guano and Oil Co.
,, (Wallsend)	Police Cells	H.M. Commissioners.
Paisley	Grain Silos	Messrs. Wm. M'Kean & Co.
,,	Starch Works	Messrs. Brown & Polson.
Peterborough (Walton)	Office Building	Messrs. Peter Brotherhood.
Plymouth	Strong Room, Post Office	H.M. Office of Works.
,,	Millbay Railway Station Extension	G.W.R. Co.
,,	Warehouse	G.W.R. Co.
Poole		Bournemouth Gas and Water Co.
Portsmouth	Custom's Watch House	H.M. Office of Works.
Preston	Stores	Co-operative Society.
Shrewsbury	County Hall	County Council.
Sligo	Park Mills	Messrs. Harper Campbell.
Southam	Cement Store	Messrs. Kaye & Co.
Southampton	Refuse Destructor	L. & S.W.R. Co.
,,	Boiler House	Southampton Cold Storage Co.
,,	Engine House	Southampton Cold Storage Co.
,,	Underground Convenience	Corporation.
,,	Cargo Shed	L. & S.W.R. Co.
,,	Shop and Warehouse	Mr. J. Hollis.
,,	Strong Room	Messrs. Thorneycroft & Co.
Stoke-on-Trent	Refuse Destructor	Borough Council.
Sunderland	Granaries and Grain Cleaning House	Messrs. R. & W. Paul.

Sunderland . .	King's Theatre . .	King's Theatre Co.
Swansea . . .	Flour Mill and Granary .	Messrs. Weaver & Co.
Truro . . .	Warehouse . . .	Western Counties Agricultural Co-operative Association.
Waterford . .	Granary . . .	Messrs. R. & H. Hall.
Wolverton . .	Schools	County Council.
York . . .	Horse Repository . .	Messrs. Walker & Co.
,, . . .	Strong Room . .	Messrs. Rowntree & Co.
,, . . .	Extension of the "Homestead"	Joseph Rowntree Village Trust.
,, . . .	Factory (two new buildings)	Messrs. Rowntree & Co.

CHIMNEY SHAFTS.

Addisham (Kent) .	Boiler Shaft . . .	Foncage Syndicate.
London (Northfleet).	Boiler Shaft . . .	Associated Portland Cement Manufacturers.
,, (Purfleet) .	Boiler Shaft . . .	Thames Paper Co.
,, (Victoria Docks)	Boiler Shaft . . .	Lyle's Refinery.

COAL STORES AND HOPPERS.

Brighton . . .	Coal Pockets, Electricity Works	Borough Council.
Cardiff . . .	Dock Coaling Pits . .	Cardiff Railway Co.
Glasgow (Yoker) .	Coal Pockets, Power Station	Clyde Valley Electricity Co.
Leeds . . .	Coal Stores, Electricity Department	City Council.
London (Neasden) .	Coal Pockets, Power Station	Underground Electric Railways.
,, (Northfleet) .	Coal Hopper . . .	Bevan's Cement Works.
,, (Royal Oak) .	Coal Hoppers . .	G.W.R. Co.
,, (Swanscombe).	Coal Hopper . . .	Cement Works.
Middlesborough (Haverton Hill)	Clinker Store . .	Messrs. Casebourne & Co.

Motherwell . .	Coal Pockets, Power Station	Clyde Valley Electricity Co.
Plymouth . .	Coal Stores . . .	Southampton Steamship Coal and Patent Fuel Co.
Portsmouth . .	Coal Stores . . .	Messrs. J. R. Wood & Co.
Rainham . . .	Coal Bunkers . .	Messrs. J. C. & J. Field.
Rochdale . . .	Coal Bunkers . .	Borough Council.
Southampton (Northam)	Coal Hoppers . .	Phœnix Wharf Coal Co.

FOUNDATIONS.

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Bath . . .	Power Station . .	Borough Council.
Bexhill (Glynde) .	Gas Works . . .	Hastings and St. Leonards Gas Co.
Bishopstoke . .	Bridge	District Council.
Birmingham . .	Power Station . .	British Electric Traction Co.
,, . .	Water Supply Reservoir .	City Council.
Bournemouth . .	Electric Power House .	Borough Council.
Bridport . . .	Tank	Bridport Gas Co.
Brighton . . .	Power Station . .	Borough Council.
Bristol . . .	Dock Warehouse . .	Docks Committee.
Cambridge . .	Fulbourne Asylum . .	County Council.
Cardiff . . .	Engine and Pumping House	Bute Dock and Railway Co.
,, . . .	Coal Hoist . . .	Bute Dock and Railway Co.
Chandlers Ford .	Pumping Station . .	South Hants Water Co.
Chatham . . .	Office Building . .	Methodist and General Insurance Co.
,, . . .	"Barracks" . . .	Salvation Army.
Coleraine . . .	Tank	Water Works Co.
Cork . . .	Brewery . . .	Messrs. Beamish & Crawford.
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Dundee . . .	Office Building .	Proprietors of *Dundee Courier*.

Edinburgh . .	Granton Gas Works .	City Council.
Felixstowe . .	Flour Mills and Granaries	Messrs. Marriage & Co.
Glasgow . . .	Chimney Shaft . .	Coventry Ordnance Works.
,, (Yoker) . .	Power Station . .	Clyde Valley Electric Supply Co.
Greystones . .	Tanks	District Council.
Guildford (Cranleigh)	Reservoir . . .	Water Works Co.
Heysham Harbour .	Power Station . .	M.R. Co.
Hull . . .	Post Office . . .	H.M. Office of Works.
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,, . . .	Wharf, Brunswick Dock.	Mersey Docks and Harbour Board.
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